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PROCEEDINGS

of the

FEDERAL INTER-AGENCY SEDIMENTATION CONFERENCE

1963

Miscellaneous Publication No. 970

Agricultural Research Service

U.S. DEPARTMENT OF AGRICULTURE

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FEDERAL INTER-AGENCY SEDIMENTATION CONFERENCE

1963

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Preface

This publication contains the papers and discussions presented at the Federal Inter-Agency Sedimentation Conference, held at Jackson, Miss., on January 28–February 1, 1963. The conference was organized and sponsored by the Federal agencies represented on the Subcommittee on Sedimentation of the Federal Inter-Agency Committee on Water Resources.

The meeting was planned as a working conference in which representatives of Federal agencies and of universities could discuss sedimentation problems and the methods being used for their investigation and solution. Through such free exchanges of information on procedures and accomplishments, duplication of effort can be avoided and a wider application of knowledge obtained, thus resulting in better coordinated programs and efforts in the field of sedimentation.

A total of 310 engineers, geologists, and others from 41 States and one foreign country attended the conference. The majority of these represented the Federal agencies. Others that participated included representatives of universities, State agencies, and organizations closely associated with the Federal agencies in their activities. A list of those in attendance is included in these proceedings.

A. L. COCHRAN, Corps of Engineers, was chairman of the Subcommittee on Sedimentation at the time of this conference. The Committees of that Subcommittee responsible for the conference include:

PROGRAM:

Chairman: CARL R. MILLER (ARS)
(Died Nov. 2, 1964.)
Co-Chairman: S. K. LOVE (GS)
Liaison: R. B. VICE (GS)

ARRANGEMENTS:

Chairman: CARL R. MILLER (ARS)
Co-Chairman: GEORGE WATTS (CE)
Special Assistants: FARRIS DENDY (ARS)
H. C. MCGEE (CE)

REGISTRATION:

Chairman: FARRIS DENDY (ARS)
Secretary: MISS JULIA MCNEELY (ARS)

PUBLICATION:

Chairman: L. M. GLYMPH, JR. (ARS)
Co-Chairman: FRANK RAINWATER (PHS)

BRICE HOBBS, Corps of Engineers, served as Conference Coordinator.

Personnel of the USDA Sedimentation Laboratory and the U.S. Army Engineer Waterways Experiment Station assisted in many ways, including local arrangements for conference meeting rooms, field trips, hotel reservations, transportation, and other facilities. Assistance during registration was provided by the Jackson Convention and Tourist Bureau.

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Italic numbers in parentheses refer to references placed at the end of each paper. The data given in the references are essentially those forwarded by the writer to the Publications Committee.

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Opening Session

INTRODUCTORY REMARKS

By Albert L. Cochran, *chairman, Subcommittee on Sedimentation, Inter-Agency Committee on Water Resources*

The subject of this conference is an important one. It relates to a problem that has a major impact on the development of water resources. We are all aware that the future prosperity of this nation or any other is dependent upon the efficient planning, design, construction, and operation of facilities required for optimum utilization of water resources. Sedimentation is a malignant growth that dissipates the effectiveness of our reservoirs, floodways, navigation channels, and harbors. It seems incurable at present, but is already responding to preventive measures in some degree. Through increased knowledge of sedimentary processes and the influences that govern these, we can avoid many errors in design and possibly direct the forces involved toward useful purposes in preserving rather than destroying lands and river improvements.

You will find that many of the papers prepared for this conference deal with current problems and immediate practical solutions, whereas others relate to research activities needed to improve future solutions. In the opening session, principal emphasis is being placed on educational and research needs.

WELCOMING REMARKS

By W. Alton Bryant, *vice chancellor, The University of Mississippi*

Mr. Chairman and Delegates:

It is a privilege to welcome you on behalf of the University of Mississippi. The University is vitally concerned with the research which you are carrying on in the area of soil sedimentation. The implications for the improvement of our economy and our society are tremendous.

The establishment a few years ago of the Sedimentation Laboratory just off the Oxford campus of the University of Mississippi is proving of great value to the University. The laboratory has brought to it first-class scientists who have contributed to the intellectual and social life of the community and who have made direct contributions to the teaching and research programs of our science departments. Since a true University is a community of scholars, the staff of the laboratory has been a real addition to our faculty.

The Waterways Experiment Station at Vicksburg, co-host with the Sedimentation Laboratory of this Conference, has over the years gained national and international recognition for technical ability and excellence in dealing with problems of control of major rivers such as the Mississippi.

I want to express my appreciation to the sponsors of the Conference, for making it possible for me to meet with you here today and have a brief part on your program. I am delighted that your itinerary will take you to Oxford and Vicksburg, where you will have an opportunity to see the University, the Sedimentation Laboratory, and the Waterways Experiment Station.

REFLECTIONS ON FIRST FEDERAL SEDIMENTATION CONFERENCE

By Carl B. Brown, *watershed specialist, Soil Conservation Service*[1]

I am deeply grateful for your invitation to make a few opening remarks at this Second Federal Inter-Agency Sedimentation Conference. You know that I am no longer a member of your professional group, although I worked for some 15 years in your field and was a party to organizing the Sedimentation Subcommittee of what is now the Federal Inter-Agency Committee on Water Resources. As chairman of the subcommittee in 1947, I had the privilege of opening the first Inter-Agency Sedimentation Conference.

In order to become somewhat better acquainted with your progress, I leafed through the annotated bibliographies prepared jointly by the subcommittees on Hydrology and Sedimentation for the years 1950 through 1958. Many hundreds of entries deal with sediment transport and sedimentation. In the early 1940's a comprehensive survey of all the existing literature, both domestic and foreign, back to

[1] Died May 5, 1963.

about 1850 on the subject of reservoir silting showed less than 150 references excluding news notes. Probably nearly that many titles, worldwide, now get into the technical literature every year on this aspect of sedimentation.

To take another point of reference, when I came into the Washington office of the Soil Conservation Service in June 1935 to direct a program of reservoir sedimentation studies, my first effort was to find out what had been done and what was known about the subject. My first discovery was that there was no compilation of existing reservoirs in the United States, much less any significant knowledge about the effects of sedimentation on them. I spent many a hot day that summer in the old offices of the Corps of Engineers in the World War I Temporary War Department Building on Constitution Avenue, which incidentally is still there, searching through the Corps files of appendices of the 308 reports to glean what information they contained. We started in that year a card index file of significant items of information on the reservoirs of the United States. It was greatly expanded by the WPA library research project, which we supervised in New York City from 1937 to 1941. This file was subsequently made available to the U.S. Geological Survey and, together with their own efforts in this field, eventually resulted in publication of a substantial inventory of data on most reservoirs larger than farm pond size.

Our literature review in 1935 turned up some results, often more qualitative than quantitative, on silting in about 40 American reservoirs, and a few in other countries. The inventory of reservoir sedimentation surveys dated 1953, issued by the Sedimentation Subcommittee, contains data on about 700 reservoir and debris basin sedimentation surveys. This inventory is now being revised by the Agricultural Research Service for the Subcommittee. The revision will contain an additional 300 new surveys, making a total of about a thousand surveys to date.

In 1935 there was only a meager collection of records of the sediment load of streams. Because of lack of understanding of the principles of sediment transport and the crude sampling equipment, many of these records were of highly questionable value. Prior to 1935 there had been only very few laboratory studies of sediment transport in this country. Most of the pioneering in this field had been done in the European universities and institutes. In 1935, the Soil Conservation Service established a Sediment Transport Laboratory in cooperation with the California Institute of Technology at Pasadena and a Bedload Research Station at Greenville, S.C. Pioneering studies at these locations were made by such present day leaders in the field as Vito Vanoni of "Cal Tech," Hans Albert Einstein and Joe Johnson of the University of California at Berkeley, Hunter Rouse of the University of Iowa, Nephi Christenson of Cornell, Alvin Anderson of Minnesota, and many others. Their early work, together with the pioneering work of Professor Lane in Colorado, Dr. Straub in Minnesota, and several able scientists at the Corps of Engineers' Vicksburg Laboratory generated a great forward push in sediment transport research.

When the first sedimentation conference was held at Denver in 1947, there was a climate of enthusiasm. World War II had been finished a little more than a year. A great national effort at water resources development appeared in the offing to offset the anticipated depression and vast unemployment that the economists of the day confidently predicted. The vast Pick-Sloan Plan had been approved for the Missouri River Basin and was getting under way. Surveys were being resumed under increased budgets in many other river basins by the Federal agencies. Also the Federal agencies were beginning to cooperate more closely than ever before under the Quadripartite Agreement of December 1943. Technical interagency subcommittees had been created to provide a mechanism for accomplishing many objectives that had not previously been possible.

The papers offered at the 1947 conference substantially summarized or at least referred to the scope and extent of our problems and our knowledge in the field of sediment transport and sedimentation. The proceedings of this conference remain a valuable document, for more than just historical reasons, in the library of anyone actively working in this field.

After the passage of 15 years, how can we assess what has happened in this field relative to needs, goals, and objectives as they were foreseen in 1947. Most of you here have been directly and intimately associated with the sedimentation field for much if not all of the last 15 years. My own assignments in the Soil Conservation Service led me progressively away from the field, and for the last 10 years I have been able to visit with most of you only occasionally. So my perspective is now from the standpoint of a program planning specialist in the water resources field who is looking from the outside toward you in the setting of what you are providing that we need for sound planning. It would be far beyond my capacity to try to present a comprehensive or even balanced review of all the pluses and minuses of the last 15 years. But I would like to leave with you three impressions and recommendations.

1. Although our knowledge of sedimentation conditions and processes has greatly advanced

in the past 15 years, as shown by the vast increase in publications, data records, laboratories, etc., it has not been commensurate with the increasing worldwide need for more sound technical information to support the development of optimum water resource plans. We are still paying every year many times the cost of needed research and investigations by adding costly factors of safety in the planning and design of our water resource improvements. There are probably two basic reasons for this. One of them has always been with us — that is the failure of administrators and program specialists to recognize sufficiently the needs for financial support because of their own lack of technical understanding. It is my impression that this situation is improving, however, because of the changing attitude in this country toward research and development. Forces outside of the field of water resource development have found it so profitable to enlarge greatly budgets for research and development that administrators in this field, as well as Congressional Committees that are concerned with appropriations for it, have been increasingly convinced that research pays.

To illustrate, the sedimentation and hydrology research in the Department of Agriculture was started in the Soil Conservation Service in 1935 with funds allocated from public works appropriations. After the projects were formulated, the initial installations built, and a highly competent group of young scientists had been assembled, we had to go before appropriations committees. Progressively, from about 1937 until after 1950, funds for this type of activity were whittled down. In 1953 the research work of the Soil Conservation Service was transferred to the Agricultural Research Service and reorganized. About this time appropriations for research and development in nuclear energy, electronics, space, medicine, and other fields began an upward spiral, the limits of which are not yet in sight. Administrators in the Department of Agriculture and members of the appropriations committees concerned with agriculture began to recognize their field of interest was being short-changed relative to progress in other fields. In this climate there has come about within the past several years a strong competition for the establishment of new research facilities. One of the earliest of these new facilities is the fine sedimentation laboratory at Oxford, which many of you will see in connection with this conference, under the direction of Carl Miller.

On the other hand, the vast expansion of research and development in the nuclear and space sciences appears to have resulted in a marked decline in the interest of graduate students in making careers in the field of sedimentation and hydrology. This constitutes one of the gravest problems of concern to this group. Heads of departments and research laboratories across the country tell me that it is exceedingly difficult to induce young engineers to take advanced studies in sediment transport and hydrology. The supply coming out of the institutions is nowhere near commensurate with future needs. With the total number of young scientists with doctors' degrees or even masters' falling far short of the national needs, this problem seems destined to become more critical. It is high time that a group such as this considers seriously ways and means by which the work in your field can be given a greater measure of glamour or other inducements that will upgrade its rating in the space age. This is my first recommendation.

2. My second impression is that we have a great storehouse of useful information that has not yet been put in a form needed by program and project planning people. Your efforts as technical specialists have been directed by administrative and other considerations largely toward (1) establishment of new research facilities, and (2) the solution of specific problems that are critical time-wise. A great deficiency has developed in the analyses and interpretation of existing data. One of the best examples, of course, is the many thousands of station years of records of sediment loads of streams all over the United States.

The Sedimentation Subcommittee is to be commended on its work of compiling statistics on the existence of these records, their duration and quality, but this alone obviously does not provide what the planner requires. Even if these individual records have been computed and published in terms of annual tonnage of sediment, the planner still has little more than an average for a generally short period of time with some indication of upper and lower limits. What could be done with this vast array of records in an Institute devoted to utilizing the most advanced statistics of probabilities and equipped with the latest types of computers needed to test various equations with respect to correlation coefficients that might be obtained? In this area I recommend that the conference give serious consideration to ways and means by which the utilization of all forms of field station and survey records may be compiled, computed, analyzed, and interpreted. This activity deserves fully as much attention as the much needed basic research on the dynamics of sediment transport.

3. Sediment transport and sedimentation need to be glamourized as a national problem. This need has been almost overlooked in the past 15 years. It should be a never-ending effort.

Do not get too deeply buried in your research and technical studies to overlook the fact that continuous publicity focused at the seats of authority and control are needed to create an adequate awareness of the requirements in your field. This is just a part of every day life in this country.

From the time the Department of Agriculture published a bulletin by the late Hugh Hammond Bennett in 1928 entitled "The National Menace of Soil Erosion," there has been no letup in the application of time and talent to the compilation, writing, and publication of useful public information on why soil erosion needs to be controlled and how it can be done. As the years have passed, this effort has been expanded to embrace the need for water conservation on farms and ranches, for better woodland management, for preservation of fish and wildlife resources, how to provide adequate recreational requirements on the rural private lands of America, and how to achieve all of these purposes through multipurpose watershed development.

This conference should consider a bulletin prepared by interagency effort through the Sedimentation Subcommittee which might have some such title as "The National Cost of Sedimentation." Pages 3 to 7 of the Proceedings of the 1947 sedimentation conference contain an outline of this kind of information. Inquiries come to the Soil Conservation Service almost every week for facts of this kind. For example: What is the annual cost and what is the annual tonnage of material dredged from the harbors of the United States? What is the annual cost of removing sediment in the purification of surface water supply? And scores of similar questions. Current news releases and bulletins coming from the Department of Agriculture are still quoting sources largely more than 10 years old.

Let me remind you also that conferences such as this serve or should serve publicity as well as technical purposes. Instead of being 15 years apart they should be held every 2 or 3 years.

Again let me thank you sincerely for your invitation to appear on this opening day of your program and to wish you well in your efforts to bring more light and knowledge into what is perhaps the most complex field in the whole area of water resource development.

SPECIAL MESSAGE

By E. W. Lane, *hydraulic engineer (retired)*[1]

Fellow scientists, I regret that I am unable to be with you. However, I welcomed the opportunity to present this message to you — especially because of our common interests and my past association with many of you. My comments, many of which are not new, deal with ideas and concepts which I think bear repeating.

The Missouri River was called "The Big Muddy" by the Indians. The Colorado River delta indicates that this river in recent geologic time has transported considerable sediment. At the time DeSoto discovered the Mississippi, the river was in flood, making it necessary to wait for a drop in stage before it could be crossed. George Rogers Clark observed a great flood on the Wabash River in Revolutionary War times. The history of Fort Pitt cites the occurrence of a record flood on the Ohio River. Even if man were able to restore all streams to their virgin conditions, large floods, transporting millions of tons of sediment, would still be a common occurrence. In most instances man's effect on the magnitude of floods, sediment discharge, and channel stability of our major rivers has been rather puny, particularly when compared with such a natural catastrophic event as was required to form the Grand Coulee by the Columbia River.

Nevertheless, we must recognize an ever increasing urgency to preserve and utilize more effectively our extremely valuable natural resources — land and water. To achieve any semblance of success, such problems as the following must be recognized and solved promptly:

1. The public and its elected representatives should be urgently advised of all aspects of water and land utilization, the need for accelerated, coordinated research efforts, and the consequences of laxity.

2. The advent of team effort must be welcomed by all concerned. The complexity of land and water resources problems, which are intimately interrelated, will require the combined efforts of our best scientists from the engineering, the physical and biological sciences, and the social science fields.

3. An overall coordinated plan must be developed to achieve the essential goals. This will require the cooperation of many Federal and State institutions and agencies, and our educational institutions, as well as international cooperation.

To put the need for additional funds to achieve success into proper perspective, consider the money spent thus far to achieve our current knowledge of rocketry, space travel, and communication. The cost of Telstar and similar

[1] Died Aug. 10, 1963.

projects alone is discussed in terms of billions of dollars. Certainly, the importance and complexity of the problems we face in the field of sedimentation are as difficult and may even be of greater significance for the survival of man.

The problems of sedimentation, which are being considered at this conference, are a significant and vital part of the broader aspects of water and land utilization discussed in the foregoing paragraphs. Many phases of sedimentation have defied rigorous mathematical analysis. The determination of correct numerical answers is difficult because of the number of variables involved. We must take stock of the results obtained and make use of these results. Insofar as possible we must avoid the use of questionable approximations in making important decisions.

Since the last meeting in Denver in 1947, notable advancements have been achieved in the areas of land erosion and control, and in sedimentation in streams, estuaries, harbors, and reservoirs. With the advent of intensified activity, increasing interest, improved instrumentation, and ever-increasing development of new methods of analysis, computer programming, and better coordination of efforts, much should be possible with regard to educating the public, improving existing theories and concepts, and conceiving new theories. Most of our studies thus far have been oriented toward a look at the average conditions and gross mechanics, using simplified systems. Much has been achieved with this approach. To advance our knowledge further, it seems essential to look more closely at the inner mechanics of sedimentation. This will require breaking the gross problem into its components and then critically examining these components.

Some of the important problems that must be studied to advance further our knowledge of sedimentation include:

1. The role of the individual variables such as velocity, depth of flow, temperature, and concentration of wash load.

2. The physical explanation of the development of bed forms, including why they develop and why they vary from ripples to dunes to plane bed to antidunes — including combinations of these. This knowledge is essential, in order to understand resistance to flow, sediment transport mechanics, and the behavior of canals and rivers.

3. The importance of turbulence and diffusion has long been discussed and recognized as having an important bearing on a more fundamental concept of sedimentation. Thus far we have been hampered by not only a lack of a satisfactory theory of turbulence, but also by a lack of adequate means of measuring its properties in water.

Other areas requiring further work of a fundamental nature include:

1. Forces on particles — such as lift and drag.

2. Improved methods of describing the significant characteristics of sediment. For example, our knowledge of fall velocity of particles is very incomplete.

3. Better methods are needed to sample sediment being transported. For example, we need to devise more adequate means of measuring contact load in alluvial channels.

In conclusion, it should be emphasized that the results of our studies are essentially useless unless they are published. Even though we may not have answered a particular problem to our complete satisfaction, our results may save other workers considerable time and expense if they are published. Finally, results should be published, using available avenues that are widely circulated, read, and respected, both nationally and internationally, that provide coworkers an opportunity not only to study the results of our work but also to discuss these results.

The foregoing statement includes ideas and thoughts which I have discussed recently and in the past with Daryl B. Simons [2] and Maurice L. Albertson,[2] who have helped me by assembling the ideas into written form. Because of my present illness, it would not have been possible for me to prepare this without their help.

KEYNOTE ADDRESS

By L. C. Gottschalk, *staff geologist, Engineering Division, Soil Conservation Service*

Floods and erosion are natural conditions which have existed throughout geologic time. With the advent of Man, the surface of the land has undergone a progressive change. Much of the natural, protective vegetation has been removed or depleted. The soil mantle has been disturbed by tillage and construction activities. Highways, streets, and buildings have been built, creating runoff from areas where formerly the rainfall percolated into the soil mantle.

With increased population comes greater demands for food and fiber and more intensive land use. Increased population means more buildings, more highways, more urbanization. The net result is greater erosion and more fre-

[2] Colorado State University, Fort Collins.

quent and higher floods and more and more sediment. History records the decline of ancient cities, even ancient civilizations, because the inhabitants could not cope with the erosion and sediment problems.

In this day and age, with our ability to produce power and machines and our technical advances and ingenuity, we no longer find ourselves completely helpless to meet Nature's assaults. But the people of the United States, the taxpayers, farmers, home owners, and industrialists, either directly or indirectly, are paying a staggering price for wasted soil and water resources. The current flood damage in the United States is estimated at more than $900 million annually, of which a sizable portion is chargeable to sedimentation. In Johnstown, Pa., after the flood of March 17, 1936, it cost $3,870,000 to remove the sediment and debris from streets and cellars. At present-day prices, this bill would be considerably higher.

Sediment accumulates in our channels, waterways, and harbors, where the cost of removal is high. Infertile sediment damages crops and reduces the productivity of our normally fertile flood plains. Sediment depletes the storage capacities of our reservoirs. At least 850,000 acre-feet of sediment is deposited in our major reservoirs each year.

The field of sedimentation has many ramifications and is complicated in scope. The geologists were among the first group of scientists in this country to be concerned about sedimentation. Their interest stemmed from a need to determine the processes involved in erosion, transportation, deposition, and consolidation of sediments to better understand the genesis and formation of sedimentary rocks. The geologists were also among the first group of scientists to be concerned about soils and about soil mechanics. But the practical solution of problems concerned with sedimentation, soils, soil mechanics, and other fields requires knowledge and skills in many different sciences in addition to geology.

Because the agricultural people needed more detailed information on soils than that which geologists could provide, a new field of soil science was developed. The same is true in soil mechanics. The engineers found it necessary to greatly expand data collection and analysis and the development of techniques and specialized equipment for evaluating the engineering properties of soil to meet requirements for design and construction of structures.

The same holds true for sedimentation. The need for more detailed information for the solution of complex problems in respect to the design and maintenance of channels, reservoirs, and other structural works of improvement has led to specialization in the field of sedimentation in the past two decades. As in other fields, the development of procedures and techniques for the solution of these problems requires studies in many disciplines. Today we have the collective efforts of geologists, engineers, physicists, chemists, pedologists, and others, all contributing to the development of knowledge needed for the solution of sedimentation problems. This is good! This is the symbol of healthy progress in any field.

Sedimentation as a field of specialization is still relatively new when compared with many other scientific fields. Important and gratifying progress is being made in this field. Today there is a much greater awareness of erosion and sedimentation problems, by both Federal and non-Federal interests, than there was when the Denver Conference was held in 1947. And today more is being done about the solution of problems than ever before.

At the time of the Denver Conference, there were no bibliographies, no inventories of sedimentation data, no standard sampling equipment and techniques, and rather limited basic data. During 1946, the year preceding the Denver Conference, sediment load measurements were being made at less than 300 measuring stations in the United States. A wide variety of sampling equipment, including soft-drink and other types of bottles, were being used to obtain samples. In 1960, sediment load measurements were made at over 800 stations in the country, using, for the most part, standard samplers and sampling procedures. There were not only more measurements in 1960, but the locations of stations gave a better representation of the different physiographic areas of the country. Furthermore, the results of these measurements are more reliable, because inherent errors due to the use of different types of sampling equipment and procedures have been largely eliminated.

The development of standard samplers and sampling procedures is a continuing function of the Federal Inter-Agency Sedimentation Project at St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minn., which is sponsored by the Subcommittee on Sedimentation. Various types of depth-integrating, point-integrating, and bed-material samplers have been designed and calibrated by this project. These are used by the Federal agencies engaged in sediment-sampling work and have been made available to non-Federal interests in this and foreign countries. The St. Anthony Falls project is now concentrating on the development of automatic equipment for sediment sampling of streams. This equipment is expected to improve further sampling results and to reduce costs. Several

pilot models are being field tested at present, and their availability for general use is anticipated in the near future.

Prior to the Denver Conference, reservoir sedimentation surveys had been made on about 375 reservoirs and debris basins in the United States. The results of many of these surveys were unpublished and were filed in widely scattered places. Surveys have now been made on more than 1,000 reservoirs in the United States. The results of these surveys are neatly tabulated and will soon be available in published form.

Today we have a radioactive probe for determinating the specific weight of submerged sediment deposits. This method is fast, relatively accurate, and does not require taking a great many samples to the laboratory for time-consuming and costly analysis. Because of this equipment, many more observations of greater accuracy can be made in less time and cost than could be made by the old method, which required obtaining questionable undisturbed samples, transporting them to the laboratory, measuring, drying, and weighing them to determine the specific weight.

As of 1946, there were more than 3,000 published papers, articles, and reports on sedimentation and related subjects. These were scattered throughout various scientific journals, periodicals, proceedings, and transactions of engineering societies and elsewhere. One man in his time could not possibly search out all of this literature, much less read it. In 1950, the Subcommittee on Sedimentation arranged for the compilation of annotations of these papers and issued Bulletin No. 2, which contains references to 3,727 papers by 2,402 different authors. These articles were classified by subject matter and by river basins to facilitate greatly the use of the bibliography. Today, there are more than 4,000 published papers, and the annotations are brought up to date periodically under the auspices of, or in cooperation with, the Subcommittee.

Fifteen years ago, we talked a great deal about the factors that influence erosion, transportation, and deposition of sediment. Today, the relative influence of many of these factors is being expressed mathematically. There are now equations for estimating rates of soil erosion, for calculating sediment delivery ratios, for computing the trap efficiency of reservoirs, and for predicting rates of reservoir silting. There are equations for deriving the ultimate sediment distribution in major and small reservoirs. Soon equations will be available for predicting rates of gully head advancement and the stability of channels in cohesive materials. Many of these equations are subject to much further improvement and refinement as more information becomes available. But, like the bed material transport equations, though still subject to further refinement, they are providing answers that are far better than the use of the crystal ball. However, with all of these advancements, the need for knowledge is increasingly emphasized, which is the primary reason for this Conference.

The Subcommittee on Sedimentation has contributed effectively to the rapid advancement of sedimentation knowledge since the Denver Conference. The Subcommittee was established by action of the Federal Inter-Agency River Basin Committee in its meeting of April 25, 1946. It held its first meeting on May 28, 1946. At that time, it took over the functions of an informal Federal Inter-Departmental Committee organized for the purpose of considering the development of sediment samplers, sampling techniques, and laboratory procedure and of coordinating the work of the several agencies actively concerned with the sedimentation problem. The so-called "Iowa Reports" were sponsored by this Inter-Departmental Committee.

The parent committee, the Federal Inter-Agency River Basin Committee, had its origin in a memorandum of agreement signed by representatives of the member agencies on December 29, 1943. By letter dated May 26, 1954, the President established the Inter-Agency Committee on Water Resources to replace the Federal Inter-Agency River Basin Committee. A new charter for a continuing Subcommittee on Sedimentation was granted by action of the new Inter-Agency Committee on Water Resources at its meeting of January 25, 1955. The Subcommittee is composed of representatives of the Corps of Engineers, Department of the Army; the Soil Conservation Service, Forest Service, and Agricultural Research Service of the Department of Agriculture; the Bureau of Public Roads and Coast and Geodetic Survey of the Department of Commerce; the Bureau of Reclamation; Geologic Survey and the Bureau of Mines of the Department of the Interior; the Public Health Service of the Department of Health, Education, and Welfare; the Federal Power Commission; and the Tennessee Valley Authority.

The continuing functions of the Subcommittee are as follows:

1. Actions to facilitate and improve efforts of the participating agencies, including cooperation with State agencies where appropriate, that pertain to the collection, analysis, and interchange of sedimentation data.
2. Actions related to clarification of technical and related administrative procedures that have an important effect on sedimentation work of

participating agencies.

3. Standardization and development of technical terminology, equipment, criteria, and methodology.

4. Exchanging information on basic and applied research in the field of sedimentation.

5. Coordination of proposals of the several participating agencies and others concerned to augment bases for national policies vital to the collection, analysis, and dissemination of sedimentation data.

6. Any other action pertinent to sedimentation and of common interest to participating agencies that cannot be effectively handled by cooperating arrangements at the field level or by individual agencies.

The chairmanship of the Subcommittee is rotated annually on a mutually acceptable basis. The Subcommittee meets at least once every 2 months with more frequent meetings if the need arises. It has no permanent staff and all work accomplished by the Subcommittee is done by financial or personnel contributions of the member agencies on an "as needed" basis.

The Subcommittee has a number of important accomplishments to its credit. It sponsors the Federal Inter-Agency Sedimentation Project at St. Anthony Falls for the development, modification, and testing of sediment sampling equipment. The results of the various phases of this work are contained in more than two dozen technical reports prepared by this project.

Each year, the Subcommittee compiles a report entitled "Notes on Sedimentation Activities," which is distributed to interested personnel within the member agencies. This report summarizes activities of the Federal agencies in the field of sedimentation and provides an opportunity for the personnel of each agency to keep abreast of the current work of other member agencies as well as their own.

The Subcommittee has issued the bibliography mentioned a few moments ago and has cooperated with the Subcommittee on Hydrology and the Geological Survey, from time to time, to keep bibliographies up to date. The latest was issued by the Geological Survey in 1962. The Subcommittee has issued an inventory of published and unpublished sediment-load data in the United States as of 1949, issued a supplement to this which includes information up to 1950, and cooperated with the Geological Survey in a recent publication which brings the information up to date as of 1960. The Subcommittee has also compiled detailed sedimentation data summary sheets for all known, reliable, reservoir sedimentation surveys in the United States and issued bulletins summarizing the data for ready reference. The first of these bulletins summarized information available up to 1950. This was revised and brought up to date as of 1953. The Subcommittee has also cooperated with the Agricultural Research Service in revising and bringing this up to date as of 1961. This publication was issued in 1964.

The Subcommittee has five continuing work groups, namely (1) Sediment in Streams, (2) Sediment in Reservoirs, (3) Sediment in Estuaries, (4) Land Erosion, and (5) Personnel Problems. At each of the Subcommittee meetings, recent reports, activities, and literature pertinent to these subjects are reviewed and discussed and the information summarized in the minutes of the meeting, which are widely distributed to personnel of the Federal agencies.

This Conference, like the Denver Conference, was planned and developed by the Subcommittee. It represents still another effort to encourage and promote the interchange of information and ideas, the interchange of procedures and methodology, of basic data, and results of basic and applied research. Much time and effort has been spent in reviewing proposed papers and in developing a balanced program. The selection of papers for oral presentation was indeed difficult, since so many good papers were proposed, exceeding almost threefold the number that could be presented within the Conference time.

The papers scheduled for oral presentation at the Conference were selected by the Program Committee in collaboration with agency representatives serving on the Subcommittee. It was the objective in this procedure to assure a reasonably well-balanced coverage of agenda topics and to reflect in the program how the interests and activities of different Federal and non-Federal agencies are constituted and related. Accordingly, the papers that are presented orally are only a sample of those prepared for the Conference and included in the Conference Proceedings.

REVIEW OF RESEARCH ACTIVITIES IN SEDIMENTATION

By VITO A. VANONI, *professor of hydraulics, California Institute of Technology*

Introduction

In reviewing research activities in sedimentation, I would like to direct my attention to the developments in recent years in conformity with the theme of this conference, namely, "Fifteen Years of Progress in Sedimentation." The trend in the activities of the Federal agencies is readily apparent by comparing the program of

this conference with that of the one held almost 15 years ago. It is clear that there has been progress in the development of methods of field measurement and in the general understanding of river mechanics. It is also clear that there is a close relationship between field and laboratory studies. The significant developments of the last generation are now integrated into the general technology of sedimentation engineering, and there is also evidence that the lag time between obtaining research results and applying them to the field is being reduced. The close association between engineering and research has also influenced laboratory programs to include studies of some of the interesting but complicated problems involving nonuniform flows in river systems.

In presenting the review of a field as extensive as sedimentation, it is impossible to cover all contributions and all aspects of the activities. The evaluation of work will depend largely on the experience of the reviewers, and no two reviewers would agree on the relative importance of any item. This presentation will cover work of recent years that has influenced the developments in the mechanics of alluvial channels. It is not intended to be complete, and several important phases of work in sedimentation and many worthwhile contributions will not be mentioned.

Turbulent Suspension of Sediment

The most significant contribution of the 15-year period preceding 1947 was the development and verification of the theory of turbulent suspension. In this short period the differential equation for suspension was derived by O'Brien (*39*) and its solution was published by Rouse (*41*) for the case of two-dimensional steady uniform flow. This has now become the classical suspended load distribution equation given by

$$\frac{c}{c_a} = \left[\frac{d-y}{y}\,\frac{a}{d-a}\right]^{z} \qquad (1)$$

$$\text{where} \quad z = \frac{w}{\beta k\sqrt{\frac{\tau_0'}{\rho}}} \qquad (2)$$

d is the depth of the stream, c and c_a are the concentration of suspended sediment at distance y and a, respectively, above the bed, w is the fall velocity of the sediment grains, τ_0 is the shear stress at the bed, ρ is the density of the water in mass per unit volume, k is the von Karman universal constant, and β is a numerical factor. This theory was first verified by Christiansen (*12*) and Anderson (*4*) in irrigation canals and natural streams, respectively, and by Vanoni (*44*) in the laboratory. These laboratory studies also brought to light the effect of sediment on the hydraulic characteristics of a stream, which can be expressed in variations of k and β. In this connection, it is pertinent to observe that the laboratory work was advanced greatly by the invention of the recirculating flume by the late Robert T. Knapp and coworkers (*27*).

The impact of the development of the theory of suspension on the general field of sedimentation studies was very great indeed. It furnished a simple expression for the distribution of suspended load that could be applied by engineers, but an even more important impact was in its clear delineation of problems yet to be solved. In surveying the research of the last 15 years, it is clear that the development represented by equations 1 and 2 affected most of the work of this period. For example, the need for more information on fall velocity called for by the equation brought forth the outstanding work of McNown and his students on the effect of particle shape (*36, 37, 38*), particle concentration (*35*), and size of settling column (*37*) on fall velocity. This same need for information on the fall velocity of natural sediment grains was met by the pioneering work of Albertson (*3*) and by the Subcommittee on Sedimentation of the Inter-Agency Committee on Water Resources (*1*). Several studies were made to determine the effect of the sediment on turbulence and the von Karman constant and on the flow in general, among which are those of Vanoni (*45*), Einstein and Chien (*19*), Vanoni and Nomicos (*46*), and Elata and Ippen (*20*). The measurements made to check the theory also pointed up its deficiencies and led to studies to improve it, such as those by Halbrun (*22*), Hunt (*23*), and Einstein and Chien (*18*). The theory also furnished a means for planning and evaluating measurements of suspended load on streams such as those made on the Missouri River at Omaha by the U. S. Army Engineers. Ismail (*25*) used essentially the same theory to study the turbulence structure of flow in a closed channel. The important discovery by Lane and others (*29*) of the dramatic effect of water temperature on sediment discharge may also be related to the development of the suspended load equation.

General Problem of Alluvial Streams

In the early 1950's there appeared a series of papers reporting research on alluvial streams in both the laboratory and field. These represented a change in emphasis from the suspension process which dominated preceding years, and a return to the approach pioneered many years earlier by Gilbert (*21*) in the laboratory and Buckley (*10*) and others in the field. However, the new generation of workers came equipped with the knowledge in fluid mechanics developed in the several intervening decades

which prepared them to clarify further the complicated problem.

In this period Einstein (*16*) published his bedload function, which applies the suspended load theory, and at the same time unites the hydraulic relations into a unified transport theory. In this work the effect of bed forms on the friction factor of an alluvial stream was accounted for explicitly by applying the relation for river channel roughness developed by Einstein and Barbarossa (*17*) from measurements on streams. Much of the value of this work lay in the fact that it pointed up the pertinent factors rather than in the relations themselves. Colby and Hembree (*14*) and others applied these relations in measuring the total sediment discharge of natural streams and in developing a relation for estimating the unmeasured load based on the Einstein bedload function and measured suspended samples. This represents a new approach, in that measured quantities such as velocity and suspended load concentration, which at best can only be estimated by present formulas, are used in theoretical expressions to estimate the unmeasured portions of the load. The resulting sediment discharge is much more reliable than one calculated from theory alone without the direct measurements. At this time also the work of Leopold and Maddock (*30*) appeared on detailed observations in streams of the variation with discharge of such quantities as sediment discharge and velocity, which served to focus attention on this interesting problem.

Laboratory studies in flumes concentrated on careful and complete observations, not only of sediment discharge, but also of friction factors, size and shape of bed forms, and their motion. Some of the contributions to this work were those of Barton and Lin (*7*), Brooks (*8*), Liu (*34*), Simons and Richardson (*43*), and Kennedy (*26*). These studies identified some bed forms not reported by Gilbert, such as the sand wave (*8*), chutes and pools (*26, 43*), and peaked antidunes (*26*), and generally refocused attention on the flow in alluvial streams as a complete system, instead of dealing predominantly with the sediment discharge, as was done in the early decades of this century. In connection with bed forms, it is pertinent to note the work of Anderson (*5*) and Kennedy (*26*) in applying classical wave theory to correlate sizes and types of bed forms instead of the Froude number alone, as done previously. The result of most far-reaching impact to come out of this flume work was that of Brooks (*8*). He demonstrated by careful experiments and reported very clearly that flow with a given depth and slope, and hence bed shear stress over a movable bed, can occur with at least two velocities. The one with the lower velocity will have a dune-covered bed, a high friction factor, and relatively low sediment discharge, and the other will have a flat bed, low friction factor, and high sediment discharge. To use Brooks's terminology, these results showed that the velocity and sediment discharge of a stream are not uniquely determined by shear stress. This tended to destroy the foundations of most sediment discharge relations that assume a unique relation between shear stress and sediment discharge.

However, a more important result of this work was the clarification of the importance of bed forms and the focusing of attention on them. By so doing, it tended to give direction to laboratory and field work much as the development of the suspended load theory had done about 15 years previously. The important work of Carey and Keller (*11*) on observation of the variation of bed forms with stage in the Lower Mississippi River, of Colby (*13*) on discontinuous rating curves, and of Dawdy (*15*) on the same subject were logical developments which drew directly or indirectly on the ideas set forth by Brooks (*8*).

While a substantial segment of the laboratory and field research of the past decade was being devoted to studying interrelations between bed forms and their effect on velocity and sediment discharge, other researchers were concerned with larger features, such as channel shape, river forms, and the characteristic meanders of alluvial streams. The work of Leopold and Maddock (*31*) on stream geometry dealt with the entire stream system. This work established that there were universal relations between the three variables of stream width, depth, and velocity and the discharge at a station, and that similar relations existed between these variables at different stations along a stream and a characteristic discharge at the station. This development was an important factor in diverting attention from details such as bed forms and focusing it on the stream system as a whole, a process that is necessary from time to time in the orderly development of a science. Other work in the general area includes that of Lane (*28*) and Leopold and Wolman (*32*) on river forms, of Leopold and coworkers (*33*), Parsons (*40*), Bagnold (*6*), and Ippen and Drinker (*24*) dealing with hydraulic resistance in curves, of Schum (*42*) on the effect of sediment-type on channel shape, and of Brush (*9*) on sediment transportation in curves.

These studies showed clearly that the resistance in curves was several times that in equivalent straight sections, and indicated that for the same shear stress the sediment discharge in a curve is severalfold less than in an equiva-

lent straight section. These two significant findings are important not only because they contribute to the understanding of alluvial streams, but because they generate a number of specific questions which will serve to organize other researches to answer them. In this sense, this work on meanders furnished the same kind of impetus as did the development of the suspended load theory two decades ago, and the clarification of the effect of bed forms less than 10 years ago.

One activity which deserves mention in any summary is the development of measuring techniques as carried out by the Federal Inter-Agency Committee on Water Resources (*2*). The impact of this activity on all work in sedimentation, whether it be research or engineering, is very significant on a worldwide basis. The fact that good apparatus and techniques are readily available for making sedimentation measurements not only makes such measurements possible in many instances, but it tends to guarantee that they will be comparable to others, and thus admissible to the fund of general information on the subject.

Other activities, such as the important work on the design of channels, scour at structures, production of sediment from watersheds, etc., have not been mentioned only because of limited time.

Summary and Conclusions

In reviewing the research work of the last 15 years, it is clear that this has been a period of great activity and significant progress. There has been a marked increase in field research on streams over previous periods and a return to laboratory studies of flows over movable beds. There has been a very apparent close liaison between field and laboratory. This is evidenced by the fact that field studies are applying laboratory results and laboratories are pursuing studies suggested by field investigations. This liaison was brought about mainly through the literature, but also, to an important extent, through the personal contact between workers and by reason of the healthy intellectual climate that exists in this activity.

I have pointed out three developments which in my opinion have keynoted the work of recent years. These are (1) the theory of turbulent suspension, (2) the clarification of the role of bed forms, and (3) the discovery of the great difference between flow in curved and straight channels. The first of these differs from the other two in that it is expressible theoretically in equations in concise and quantitative form. As such, it is readily understandable, and can be assimilated into textbooks and preserved permanently. The other two items are qualitative ideas, expressible only in words, and hence their true significance is appreciated only by those with some familiarity with sedimentation. Because of this they are more difficult to incorporate into textbooks and are in danger of being lost and then rediscovered, as was the work of river engineers of several generations ago.

I believe that these examples serve to illustrate the importance of theory, and the urgent need for its further development in sedimentation. Theory forms a framework into which information can be fitted or filed in such a way that it is readily available. It not only tends to collect bits and pieces of information from old files by providing the filing system, but it also stimulates workers to get information needed to complete the picture that the theory was intended to portray.

The theoretical treatment of the sedimentation problem is very difficult, and will develop slowly. It will be based on the understanding gained from experiments rather than by some break-through by a stroke of theoretical genius. However, in order for the experiments to contribute understanding, they must be designed carefully to answer certain questions or to prove or disprove hypotheses based on reasoning and results of other investigations. Considering the primitive state of knowledge of sedimentation, contributions can be made in many ways. Clarification of old ideas based on published data can be of significant value and publication of data, particularly from the field, showing peculiar behavior of streams, can contribute to general progress even if the author cannot explain what he has observed. In a sense, the understanding of sedimentation, which we are all seeking, can be likened to a structure. The plans for this edifice will be the theory that we ultimately hope to develop to express our understanding of sedimentation processes but which is not yet available. The bits and pieces of information resulting from researches are the bricks for the edifice and will all fit into it in an orderly way once the plans are complete. In the meantime, they must be stored, i.e., published, or they are in danger of being lost to future builders of the sedimentation edifice.

Before concluding my remarks, I would like to say a word about the future. With the growing population and the increasing demand for water resources, it is clear that sedimentation will assume even more importance than it has in the recent past. This will generate the need for better solutions to sedimentation problems which must be based on better understanding of the basic processes of sediment movement. The population and the demand for water are exploding, while the research activities in sedimentation are only moving along at a fast walk.

Since the research results of one generation tend to become the working tools of the next, it is clear that it is urgent to accelerate the pace of research. To do this we need three things: (1) leadership, (2) money, and (3) research workers. The leadership must come from engineers and other technicians responsible for the water resources development. They are the ones that will be faced with the problems and the responsibility for their solution. It is also their responsibility to see that means are available for coping adequately with problems to come. But it is also the responsibility of the experts in the profession, as represented by this group, to call attention to the need for the added activity so that funds and facilities will be provided.

Without a competent well-trained body of research workers, the program will not progress and it will be impossible to continue its financial support. To train a research man takes several years of formal study and additional years of experience in research. A number of universities offer graduate work in fluid mechanics and sedimentation that will prepare people for research in sedimentation, although the enrollment in such programs is less than can be accommodated. The bright young man is turning his back on sedimentation and river engineering in preference to the fashionable pursuits of the space age. Without such people, we will not be able to meet our obligations in the years to come. This situation can be changed by a vigorous campaign to inform people of the important, fascinating, and challenging problems in sedimentation and of the opportunities that this field provides for careers. All of us can help in this task. We can accept invitations to talk about our work to engineering groups, clubs, and particularly to high school groups and individual high school students. We have an important, interesting, and challenging profession, but we need to let the world know about it.

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EDUCATIONAL NEEDS AND OPPORTUNITIES IN THE SEDIMENTATION FIELD

By M. GORDON WOLMAN, *geographer, Johns Hopkins University*

The subject of this paper, needs and opportunities in education, cannot really be divorced either from the preceding papers by speakers of very much more experience in this field than I or from the subjects which will be discussed on subsequent days at this meeting. Nevertheless, I will try here to take a rather general look at some of the educational approaches to diverse aspects of the field of sedimentation.

My method consists of a crude demand and

supply analysis, including a statement of the compass of the subject, an estimate of the practitioners, and an evaluation of the educational needs of several different groups.

Like hydrology, it seems somehow to be the custom to consider sedimentation solely as a kind of subspecies of a variety of engineering fields. This is a narrow view, however, and interest in sedimentation is actually much broader than this. Archeologists study history from depositional sequences. The interests of geologists range from surficial processes to detailed analysis of the structure and fabric of deposits. Within the broad field of engineering itself, sanitary engineering deals with sediment as a pollutant and with the ecologic effects of sediment; soil mechanics, with the mechanical behavior of sediments; civil engineering with river control, reservoir design, harbor development, and with all types of construction. Foresters are concerned with land use. Agronomists and others in land management are similarly interested in sedimentation from the standpoint of conservation of the land and increasing yields from the land.

Unlike the dictionary, which refers to sedimentation as deposition of sediment, it is clear that these varied approaches to sedimentation, indeed this conference itself, include within the subject erosion, transport, and deposition. As I will attempt to show, this threefold division, perhaps because of its very neatness, appears to have created some surprising barriers. One of the educational tasks is the destruction of such barriers.

Although the interests mentioned are quite extensive, it is much more difficult to discover the number of persons actually engaged in the field of sedimentation. The 1960 survey of scientific personnel by the National Science Foundation indicates that there are only 500 scientists in this country who consider hydrology as their first specialty. About 100 indicated a primary specialization in civil engineering aspects of waterways and harbors. Although this presumably does not include a fairly large contingent of engineers, it seems apparent that if there are so few hydrologists, the number who would consider themselves specialists in the field of sedimentation is very much smaller.

Looking, for example, at the program for this meeting, I note that there are 91 papers and abstracts. Assuming perhaps 1½ authors (it is usually the one-half author who does most of the work), we might tentatively conclude that there are perhaps 150 "sedimentationists" generally associated with the research aspects of sediment studies. The majority of these are in fact more occupied with practical problems relating to design and operation of hydraulic works than with research per se. Nevertheless, these authors represent a large segment of the professionals specializing in sedimentation.

At the opposite end of the scale it must be observed that soil conservation districts in the United States embrace more than a billion and a half acres, a vast area. In addition, hundreds of large reservoirs and thousands of smaller ones have been and are being constructed today, long sections of river channels have been canalized, and harbors cleaned of sediment. All of these activities involve problems in sedimentation. It is quite clear then that if one takes a possessive view, there must be quite a large number of professionals who are "poaching" on the field of sedimentation. Let us guess that there are several thousand so involved.

The probable population of those directly connected with engineering aspects of sedimentation might be closer to 1,500. Within the teaching profession itself, the number of professionals engaged in teaching and research in sedimentation can only be guessed. As I will show in a moment, this number must in fact be exceedingly small. Thus, I would surmise that in the field of sedimentation as a whole, we are probably talking at most about some 2,000 people in this country. (No doubt there are a larger number following various manuals in attempts to make designs for the kinds of engineering problems in which sediment enters in.) The number of true devotees must be closer to 200 to 300.

Judging from the number of geologists who selected sedimentary petrology, petrography, or recent and paleo-sedimentation as their first specialty in the manpower survey, there are perhaps an additional several hundred geologists actively pursuing interests in the field of sedimentology.

It is questionable whether one can sum these diverse interests, but as a rough estimate let us assume that there are about 1,000 professionals genuinely interested in sedimentation.

Despite some sad previous experience with attempts to analyze college courses in hydrology from either college catalogs or from the response to questionnaires sent to educators, I tried to compile some information on the status of courses in what might be called sedimentation from a sample of 30 or 40 college catalogs. Courses in civil engineering and in geology were reviewed.

Although I purposely chose the larger State universities as well as the bigger private institutions whom I supposed might include offerings in this field, confirming what most of you already know, it appears that there are perhaps 10 universities in the United States that include formal courses in their engineering schools dealing with problems of sediment transport or with the broad field of sedimentation. These are

also the universities which have large hydraulic laboratories in which work in sedimentation may be pursued. Only seven perhaps have really concentrated in the field of sediment transport. This sample does not include the agricultural programs in which sediment and erosion problems are most certainly mentioned.

My review of college catalogs indicated that there are about a dozen formal courses which carry titles such as "Sediment Transport," or "Problems of Sedimentation," or "Engineering Aspects of Sediment Problems." Several universities have specific courses related to coastal engineering, rivers and harbors, or irrigation and canal design.

(I should emphasize here that throughout this review, I am not lamenting, I am simply reporting.)

Within the field of geology, there are many more courses in the broadly defined field of sedimentation. These include stratigraphy, sedimentary petrology, and sedimentology. Virtually all geology departments offer stratigraphy, less, but most, offer sedimentary petrology, but relatively few offer courses in sedimentology. Although in the general field of sedimentation, the orientation of all of these courses is toward distinguishing environmental or stratigraphic characteristics of deposits and rocks. Sedimentology comes closest to being a course in sedimentation and this course is the least common of the group.

The present status of education in sedimentation is perhaps most clearly illustrated by a brief look at some of the textbooks in the field. In the engineering aspects of the subject coverage usually consists of several chapters in general texts on hydrology or hydraulics. One of the more complete coverages, from the standpoint of subject matter included, is the chapter by Brown in the volume *Engineering Hydraulics* edited by Hunter Rouse. This volume is much in demand and I believe out of print. The chapter on sediment transport includes material on sediment yield, reservoir deposition, transport, river control, beach and harbor problems, and sediment sampling. The treatment is necessarily short but the subject matter is broad. A number of books, many quite old, do deal with river behavior, but most are rather specialized and do not emphasize principles of sedimentation.

For the geologically oriented sedimentationists, there are a variety of texts beginning with Twenhofel's *Treatise on Sedimentation* and including such excellent works as Pettijohn's *Sedimentary Rocks* and a number of other texts in stratigraphy and sedimentation, in sedimentary petrology, as well as a few in geomorphology such as the classic *Physics of Blown Sand and Desert Dunes*. Somewhere between the engineering and geologic approaches is the series of collected essays on *Applied Sedimentation* edited by Trask. All of these offer material of tremendous interest, but due to the highly specialized treatment of each, they do little toward providing a comprehensive view of the field.

Both from the standpoint of courses and texts, the present condition of education in the field of sedimentation can be characterized rather simply. First, the quantity of offerings is limited. Second, the material offered is generally fragmented or for a specialized audience appropriate to particular aspects of individual disciplines. Lastly, virtually no courses or texts are available that present a comprehensive view of principles and problems in sedimentation traversing the entire "sedimentary" cycle from erosion through transport to deposition and lithification.

Having looked very briefly then at the present state of affairs, let me consider equally briefly some specific needs and opportunities. The educational demands may be conveniently separated into four parts: (1) General education; (2) design and practice in engineering applications; (3) specialized disciplines; and (4) research.

General education in principles of sedimentation applies to a much larger audience than we are accustomed to think of. In this I include introductory courses in natural resources, conservation, ecology, and general geography. The variety of offerings are enormous, and they range in level from grade school to graduate school. Erosion and sedimentation are dealt with as aspects of land use and management. As a result of the very successful efforts begun by Hugh Bennett and others in the 30's most of these courses and texts treat the subject with warmth, if not alarm. One might say that the present need is met satisfactorily. Perhaps if anything is needed, it is a more thoughtful approach that outlines the processes involved and provides the student with a better framework than the citation of isolated catastrophic examples usually provides. In addition, some information needs updating and clarification.

(My sanguine view that saturation by conservation concepts is attained is probably too optimistic. It may be doubted, in fact, that many citizens in our large urban communities are either aware of the nature of conservation or of the concept of intelligent land use.)

The second category, education in sedimentation for those engaged in engineering enterprises has some aspects of general education in that perhaps the largest single task is to make the engineering profession, and particularly the civil engineering profession, fully aware of the

significance of sediment behavior. There are still a great number of practicing engineers who are either unaware that sedimentation poses certain special problems in their field or who choose to ignore the problems of which they are aware. Relatively few texts in open channel hydraulics consider the kind of flow problems, which will be discussed at this meeting, that are related to water and sediment movement in channels with mobile boundaries. The same could be said about the treatment of a variety of other engineering aspects of sedimentation. One of the missions then for education in the field of sedimentation would seem to be that of increasing the awareness of the field on the part of all of the practitioners. It is unlikely that most engineers will need to or will be likely to become experts in sedimentation alone. However, they share a need for a comprehensive understanding of the field of sedimentation with foresters, geographers, agriculturalists, geologists, and others concerned with land and water problems.

To satisfy this demand for a comprehensive introduction to principles of sedimentation, formal courses and texts are needed that consider the entire erosion cycle pointing up the interconnection between the parts and emphasizing the unanswered problems. Hopefully, such courses would include the dynamics of erosion, the mechanics of transport by various media, and the nature of depositional environments. The amount of detail with which each subject would be covered would depend upon both the length of the course and the degree of specialization of the audience. Such a course would not necessarily be a substitute for specialized subjects such as sediment transport and canal design. It would insure a general introduction, however, both for those who will receive the specialized courses and for others. I would hope that eventually all the universities and colleges offering major programs relating to water and land use would include such a comprehensive course.

In connection with the stimulation of awareness of the principles and problems in the study of sediment, an outstanding educational need lies in disseminating information to areas now only beginning to develop their resources and economies. Here the comprehensive view is essential. Many such places in humid tropical regions, for example, will face complex sediment problems as watersheds are cleared, land developed, and dams constructed. My own travels have convinced me that as we become more sophisticated and specialized, it becomes harder for us to communicate with these areas of the world. Many have little or no access to most of the published literature in the field.

Because I consider it one of the most useful services that the United States might provide in this field, I should like to suggest that this Federal Inter-Agency Sedimentation Conference take the lead in preparing an outline and in assigning authors to portions of a general manual in the field of sedimentation. (If it didn't have the wrong connotation, I would characterize it as "a poor man's manual of sedimentation," much as many of us refer to the need for "a poor man's 888," to supplement the U.S. Geological Survey manual of stream gaging. My model would be something like the handbook, *Low Dams,* or perhaps some portions of *Low Dams* grown bigger, called "Small Dams.")

The approach of such a manual would be on the "how to decide what to do," and "how to do it" level. For this reason, it might borrow heavily from existing manuals of the Federal agencies and from the American Society of Civil Engineering manual that is in preparation, but the sedimentation manual should differ from the others in style and content. Topics covered might include sediment descriptions, alternative methods of sampling sediment in streams and principles of network design, problems of scour and fill, estimating sediment yields, sediment delivery ratios, measuring sediment in reservoirs, and erosion controls. Suggested methods in each of these topics would include the most simple techniques, as well as the usual fancy ones, and present selected data that would emphasize evaluation of the costs of alternatives, and discuss the appropriateness of certain techniques to specific regions or to specific problems at desired levels of accuracy. This might be helpful in combating the conflicting tendency encountered not infrequently; that is, precise measurement of very little at great cost in labor and equipment, or no measurement at all on the assumption that all observations must be made with complex equipment and by highly educated personnel.

Virtually all of those who could compile such a manual will attend this meeting. Perhaps a start can be made. I should guess that it could be completed in 6 months to a year. Let me turn now to the two remaining areas of education in sedimentation; specialized disciplines and research.

Physical geologists and geomorphologists are primarily concerned with landscape, with erosion and transportation of sediments, and, on occasion, with particular depositional forms. This interest in landscape and process leads to a direct concern with the broad relations between climate, erosion, and deposition, and thus also to concepts of equilibrium and climatic change in the evolution of land forms. These

questions in turn are of direct consequence to professionals concerned with land management. Elsewhere within the field of geology, however, stratigraphers are concerned with the characteristics of sedimentary deposits that have become rocks. There is considerable current interest in attempts to compare modern sedimentary environments with paleo-environments, particularly efforts to understand the sedimentary conditions associated with the location of oil. Unfortunately, relatively few of those who study modern sedimentation also look at the rocks and vice versa. Here again there is a real need for a true combination in which the dynamics of modern sedimentary processes and depositional environments are analyzed and described in a comparable way such that features can be seen in the rocks and interpreted.

Such courses are undoubtedly taught in a few places, although I am not aware of many of them. Nevertheless, although there is yet a great deal to be learned, a text which approached the field from this standpoint would be useful in orienting students to the field. It would treat the mechanics of various kinds of transporting media, their sorting effects, the products derived from such processes, and environmental controls of deposition.

Much has already been said here today regarding the fourth aspect, research. It is perhaps first in importance. Because training for research is problem oriented, broad generalizations cannot be made about the kinds of education that would be appropriate unless the problem is specified. A scientist interested in the mechanics of erosion of clay may need a great deal more work in theoretical chemistry, clay mineralogy, and soil physics than will a scientist interested in some aspects of sediment transport. Although many unanswered questions in the field of sedimentation remain, well-trained people will not be attracted to the field unless these challenges are brought to their attention. Similarly, as Professor Mason noted at the first Federal Inter-Agency Sedimentation Conference, potential supporters of research must be made aware of the gaps in knowledge and of the need for research. This is a common responsibility. It requires an admission that all is not known. Further, it is up to the educators to stimulate the interest of their students in the unknowns. Research not only provides a means of teaching, but it also enhances this consciousness of the unknown and thus the needs of education are also those of research. The Federal agencies can do much to create an awareness of research needs in Congress and with the public at large, although this will require an occasional admission that all the answers are not known.

In summary, then, it would be a mistake to assume that because many of us gathered here share an intense common interest, that vast numbers of experts in the field of sedimentation will be required in the future. In this category alone, I should place the total between 200 and 400 individuals at most.

Additions and replacements in this group, perhaps 10 to 20 per year, will require concerted efforts — not because the number is large but because today the competition for all kinds of scientific personnel is stiff.

Judicious emphasis on, and support of, research should stimulate university interest and serve to attract students to the field. Numerically, a larger task of education would appear to be in broadening the understanding of sediment problems among engineers and others dealing with land and water. These two complimentary objectives will require some increase in faculty qualified to teach principles and problems in sedimentation. Hopefully, more of those now being trained to such competence will remain in the teaching profession and will not be wholly absorbed by engineering firms in which their specialized interest is little used.

In a word, we need a comprehensive view, we need courses which expound that view and textbooks that facilitate such courses. And to keep the field vital, we need strong research efforts to ask questions and hopefully to answer some.

Lastly, there is an educational responsibility to disseminate what we have learned to a wider audience. Not necessarily because we know a great deal more but at least in part because we have been able to afford a wide range of experience, experience which has included howling success and occasional dismal failures. Both should be reported.

Hopefully, this has not been entirely platitudes but has given you something to feed on.

Symposium 1.—Land and Erosion and Control

INTRODUCTION

That many of the sedimentation problems encountered in water resource development programs are related to the rates and magnitude of erosion on watershed lands and in stream channel systems is the dominant theme of Symposium 1. Of the 26 papers included in Symposium 1, 14 deal with subjects of sediment sources and yield, 6 with erosion control, and 6 with miscellaneous topics somehow related to the origin of sediment, the measurement of sediment, and the factors affecting sediment genesis and movement.

The effects of land use and management practices upon runoff, erosion and sediment yield are discussed, including erosion control and conservation practices on agricultural lands, forest cover and logging practices, and the use and management of range lands. The significance of gullying and stream channel erosion as sources of sediment is indicated, and practices for controlling these sediment sources are mentioned.

Sediment problems associated with highway construction and urban housing and development projects are likewise pointed out. Factors affecting gullying, stream channel morphology, and the shear resistance of cohesive sediments are treated.

The papers in this Symposium point to the complexity of interrelations between erosion and related sediment problems. They do not contain all of the answers, but they are positive and optimistic by showing that much has been learned about these interrelations and that erosion control practices properly applied and maintained on watershed areas can be an effective means for dealing with many sediment problems.

EROSION AND ITS CONTROL ON AGRICULTURAL LANDS

[Paper No. 1]

By JOHN W. ROEHL, *geologist, Engineering and Watershed Planning Unit, Soil Conservation Service*

Erosion is the wearing away of the land surface by detachment and transport of soil and rock materials through the action of moving water, wind, or other geological agent. Normal, or geologic, erosion pertains to this wearing away of the land under natural environmental conditions. Accelerated erosion, brought about by man, is conceived to be the erosion occurring at a rate greater than normal for the site, usually through reduction of a vegetal cover. While both wind and water are the principal agents producing accelerated erosion, this discussion will be confined to the water-caused process.

The interest in erosion and its control on agricultural lands is twofold. First, unchecked accelerated erosion represents a loss of a natural resource upon which we depend for our food and fiber; and, second, its end product — sediment — can represent a damage to flood plain lands, to urban areas, to navigation facilities, to downstream reservoirs, and to other improvements.

As defined, the erosion induced by improper use of the land proceeds at a rate greater than that which should be expected under purely geological conditions. It is this accelerated rate of erosion in which our main interest lies, as it is the rate that can be altered. If the accelerated rate through the application of various measures on agricultural land is reduced, our basic soil

FIGURE 1. — An example of sheet erosion with accompanying rilling and deposition.

resource can be maintained and the production of sediment can be minimized.

Accelerated erosion can be classified into two main types: sheet and channel erosion. Sheet erosion is the more or less uniform removal of soil from an area without the development of conspicuous water channels. Included with sheet erosion, however, are the numerous small but conspicuous rivulets, or rills, that are caused by minor concentrations of runoff. The rills are easily obliterated by normal field cultivation. Figure 1 portrays an instance of sheet erosion. Channel-type erosion is that caused by the concentrated flow of water. Gullies, valley trenching, streambed, and streambank erosion are included in this latter type. A well-developed gully is shown in figure 2.

FIGURE 2. — An example of channel-type erosion.

The importance of the two types of erosion, both as to the loss of a resource and as a source of sediment, varies widely in different areas of the country. It can be stated fairly safely that in the humid part of the country sheet erosion is the prime offender, whereas in the more arid parts, where rainfall is experienced in short, high intensity storms, channel-type erosion is the source of greatest soil loss and ultimate sediment production. However, in small areas, as well, the relative importance of each type can be markedly different.

To pinpoint these generalities, erosion estimates obtained in the course of watershed planning in the Southeastern United States were analyzed as to the importance of the two types.[1] This analysis, involving some 1,720 square miles well scattered throughout the Southeast, indicated that sheet erosion accounted for the greater part of the computed erosion; ranging from 66 to 100 percent of the total. In four major land resource areas, sheet erosion accounted for essentially all of the computed erosion. This is in terms of rather broad areas.

To show that there is a wide disparity in importance between the two types within small adjacent areas, data obtained by the Agricultural Research Service[2] in the Pigeon Roost Watershed of northern Mississippi was analyzed. This analysis indicated that, in contributing watersheds of 0.2 to 23 square miles in size within a total watershed area of some 65 square miles, sheet erosion comprised 47 percent to over 90 percent of the total or gross computed erosion.

In terms of sediment yield, it is probable that, in the humid areas, sheet erosion accounts for the greater part of the total sediment load. Due to the processes of erosion, materials derived from sheet erosion sources are the fine-grained materials swept from fields and carried in suspension to and through the conveyance system. Owing to the small particle sizes there is little opportunity for deposition until an area of slack water (i.e., a downstream reservoir) is encountered. Channel-type erosion generally is the source of larger grained materials, and this material is obtained from areas already a part of the transportation system. Thus, from the viewpoint of damaging sediment (infertile overwash, stream channel fill, etc.), channel-type erosion may be of more importance than the sheet type.

If the sediment delivery ratio is assumed in terms of the total erosion, the measure of importance of each type of erosion as a source of sediment will be the same as its distribution within the several types. It is probable, however, that the delivery of erosional debris from each type is different, but to what degree is a matter of continuing and needed study.

Many means are employed on agricultural lands to control accelerated erosion. Basic to almost all efforts for the reduction of erosion is land treatment. This is accomplished mainly through the improvement of protective cover on the soil surface exposed to the eroding forces. This improvement in cover conditions is achieved by the establishment of various agronomic and forestry practices. The capability of the land to support a desired agricultural use with a minimum amount of erosion must be considered in the application of land treatment measures. In its most simple sense, one might express this philosophy as the growing of clean-tilled crops on the level, least erodible land and

[1] ROEHL, J. W. SEDIMENT SOURCE AREAS, DELIVERY RATIOS, AND INFLUENCING MORPHOLOGICAL FACTORS. Internatl. Assoc. Sci. Hydraulics Commission of Land Erosion Pub. 59, pp. 202-213. 1962.

[2] SPRABERRY, J. A., WOODBURN, RUSSELL, and MCHENRY, J. R. SEDIMENT DELIVERY RATIO STUDIES IN MISSISSIPPI, A PRELIMINARY REPORT. Agron. Jour. 52: 434-436, 1960.

of hay, pasture, and forest crops on the steep erodible slopes. Inherent properties of soils, however, as well as the landscape, do not permit such a simple solution.

Land treatment measures in conjunction with supporting mechanical field practices are effective in controlling accelerated erosion. First, consider the measures that improve the protective cover, such as:

(1) Conservation cropping systems that encompass the growing of crops in combination with needed cultural and management systems. The cropping systems involve the use of rotations that may include grasses and legumes grown in desirable sequence. Such systems do much to maintain fertility and good structure in the soil, assisting the soil to resist erosion.

FIGURE 3. — Severely eroded area, formerly cultivated, that would be classified as a critical area.

FIGURE 4.— Stabilization of critical area, shown in figure 3, accomplished by the establishment of trees (in background), lovegrass (in center), and kudzu (in foreground).

(2) Cover and green manure crops are crops of close-growing grasses, legumes, or small grain used primarily for summer or winter protection. The crop usually occupies the land for a period of 1 year or less and provides resistance to erosion during periods when the soil surface would otherwise be bare.

(3) Critical area planting that is done to establish vegetative cover on severely eroding and silt-producing areas. Stabilization of such areas can be accomplished by the planting of woody vegetation or the seeding or sodding of adapted grasses or legumes to provide long-term ground cover. (Figs. 3 and 4.)

(4) Hay land planting that provides protective cover through the establishment of long-term hay stands of grasses or legumes.

(5) Crop-residue use that utilizes plant residues left in cultivated fields by incorporating them into the soil or leaving them on the surface during that part of the year when critical periods of erosion usually occur.

(6) Mulching that involves the application of plant residues or other suitable materials not produced on the site to the surface of the soil.

(7) Pasture planting in which adapted species of perennial, biennial, or reseeding forage plants are established on new pasture lands converted from other uses.

(8) Tree planting that includes the planting of tree seedlings or cuttings in open areas to establish a stand of forest trees.

(9) Woodland interplanting that is the planting of tree seedlings in sparsely or inadequately stocked stands.

(10) Various management practices, such as the proper use of pastures and ranges, deferred grazing, stubble mulching, that are used to maintain protective cover to the land.

Other measures, similar to those just listed, are used, but the point for consideration here is that these are agronomic and management practices employed to establish or improve the protective cover afforded by vegetation as a means of reducing erosion.

In addition to and in conjunction with the practices just discussed, mechanical field practices are usually employed. While some of these practices are more pronounced in their effect on erosion than are others, all of them afford additional reductions in the rate of erosion. Some of the more common mechanical field practices recommended for use are:

(1) Contour farming which is the conduct of farming operations on sloping, cultivated land in such a way that plowing, land preparation, planting, and cultivation are done on the contour. Depending upon the steepness of slope, erosion from contoured fields may be from 100 percent to 50 percent of that expected from up

and down the slope tillage.[3] It is most effective during low-intensity storms.

(2) Contour stripcropping that involves the growing of crops in a systematic arrangement of strips or bands on the contour. The crops are arranged so that a strip of grass or close-growing crop is alternated with a strip of clean-tilled crop or fallow. This practice works well in conjunction with conservation cropping systems, especially where a grass or legume is included in the crop rotation, and is effective in reducing erosion and sediment yields. Erosion from contour stripcropped fields averages from 25 to 45 percent of that expected from up-and-down tillage (fig. 5).[3]

FIGURE 5. — Contour stripcropping.

(3) Contour furrowing on range or pasture land is accomplished by plowing furrows on the contour at intervals varying with the slope and ground cover.

(4) Gradient terraces are established by the construction of an earth embankment or a series of ridges and channels across the slope at suitable spacings and with accepted grades. These are applied to reduce erosion and sediment yield by intercepting surface runoff and conducting it to a stable-outlet at a nonerosive velocity. Contour farming goes hand-in-hand with terracing (fig. 6), and stripcropping often is used in conjunction with gradient terraces.

(5) Level terraces are also a series of ridges and channels constructed across the slope at suitable spacings, but they have no grade.

(6) Diversions are graded or dug channels, with a supporting ridge on the lower side, across the slopes and are used to divert water from areas where it is in excess to sites where it can be used or disposed of safely. This mechanical aid is often used to intercept water from higher elevations before it can reach and erode cultivated fields.

(7) Grassed waterways are natural waterways or depressions that are reshaped or graded and on which suitable vegetation is established. They provide for the disposal of excess surface water from terraces, diversions, or from natural concentrations without damage by erosion.

All of the foregoing discussion of the land treatment measures for controlling erosion has been concerned with those measures aimed primarily at sheet erosion, although the control of minor gullies can be accomplished through their use. The control of channel-type erosion, which would include larger gullies, streambank cutting, valley trenching, etc., usually requires more than the establishment of vegetation. Structures needed may involve earthwork, or the construction of works built of concrete, masonry, metal, or other materials as listed below.

(1) Check dams can be placed in active gullies to reduce downcutting and widening. The effect is accomplished by providing areas of deposition behind the structures, thus reducing the bottom gradient. The structures can be of a temporary nature, constructed of brush or logs, until permanent vegetation finally heals the gully, or

FIGURE 6. — Gradient terraces and contour farming.

FIGURE 7. — Process of gully stabilization by the construction of a dam (in background) and the planting of bermudagrass (in foreground).

[3] AGRICULTURAL RESEARCH SERVICE. A UNIVERSAL EQUATION FOR PREDICTING RAINFALL-EROSION LOSSES. U.S. Dept. Agr., ARS 22-26, 11 pp., Washington, D.C. 1961.

they can be of a more permanent nature, constructed of masonry or concrete, if it is believed vegetation will be too long in establishing itself. Figure 7 shows a gully in which a dam has been constructed to reduce the bottom gradient and to assist in stabilization until the grasses planted on the actively eroding areas provide protection. Not only do the areas of deposition afforded by the structures aid in the control of the gully, but they also are quite effective in reducing the sediment yield from the gullies by trapping the greater part of the eroded material.

(2) Where tributary streams or ditches convey water into larger streams or canals, headward erosion often occurs. In order to reduce the erosion and possible loss of agricultural land, drop structures can be used. Such structures, of concrete, masonry, cribbing, or sheet piling, provide the means of conducting the water through differentials in elevation without downcutting in the tributaries.

(3) Streambank stabilization is sometimes required when the loss of productive flood plain land is occurring. Some of the means employed to prevent this erosion are bank sloping, riprap, deflectors, and jetties.

(4) Gradient control structures have been employed to reduce the upstream migration of valley trenches, thus eliminating the loss of land caused by such erosion. To be effective, this type of structure should be constructed so that the active headcut of the valley trench is covered by the impounded water.

That the management and agronomic practices enumerated earlier are effective in reducing the rate of accelerated erosion is well borne out by research. Crop rotations that include a grass or legume have been shown to reduce the rate by about 70 percent of that experienced under fallow or completely unprotected conditions. The establishment of dense hay and pasture grasses can similarly reduce the rate of soil loss to practically nothing.

Depending upon the steepness of the slope, the addition of mechanical field practices will reduce the rate still further. Contouring, alone, will afford a reduction in erosion rates from 10 percent on the steeper slopes to 40 percent on the flatter slopes. Contour stripcropping on terraced land will accomplish reductions of 55 to 70 percent in soil loss, again depending upon the steepness of the slope.[4]

It follows, quite properly, that if the rate of erosion in an area is altered, the rate of sediment yield from that area will also be changed. An example of what conservation can do in reducing sedimentation was experienced in Lake Issaqueena, S. C., where it was found [5] that the establishment of conservation measures and increased vegetation resulted in a decrease of 37 percent in the rate of sheet erosion and of 52 percent in the rate of sediment inflow. The reduction in the amount of sheet erosion was responsible for a large part of the reduction in the sediment inflow, the remaining reduction being due to the stabilization of gullies and eroding roadbanks.

It is upon this basic premise of reducing sediment yield through the application of proven conservation measures that the Soil Conservation Service predicates its sediment design for structures requiring sedimentation features. Not only is sedimentation reduced, but also our irreplaceable soil resource is maintained to continue its role of producing our needed food and fiber.

[4] U.S. DEPARTMENT OF AGRICULTURE. SOIL LOSS ESTIMATION IN THE SOUTHEAST. Agricultural Research Service — Soil Conservation Service Workshop, Athens, Ga., Apr. 12-13, 1960, 38 pp., mimeo. 1960.

[5] NOLL, J. J., ROEHL, J. W., and BENNETT, JACKSON. EFFECTS OF SOIL CONSERVATION ON SEDIMENTATION IN LAKE ISSAQUEENA. U.S. Dept. Agr., SCS-TP-95, 20 pp. 1950.

SOME INTERPRETATIONS OF SEDIMENT SOURCES AND CAUSES, PACIFIC COAST BASINS IN OREGON AND CALIFORNIA

[Paper No. 2]

By HENRY W. ANDERSON, *project leader*, and JAMES R. WALLIS, *research forester, Water Source Hydrology Project, Pacific Southwest Forest and Range Experiment Station, Forest Service*

Differences in sediment discharges from watersheds can be attributed to differences in erosion from watershed slopes and channels and to differences in the transport of eroded material from the watersheds. The rates of sediment discharge from watersheds represent an integrated average of the sediment discharge from all parts of the watershed. The rates from the individual parts of the watershed — the sediment sources — may be evaluated by analyzing data from many watersheds. Between watersheds, the discharges of sediment vary in response to differences in streamflow, soils, topography, and land use. Within the parts of single watersheds, similar responses occur. Thus, sediment sources and causes can be expressed in terms of sediment production from whole watersheds as related to variables of the meteorological potential, topographic potential, soil erodibility, and land use and condition.

This paper reports the differences in sediment discharge associated with specific measures of these potentials largely from our studies of sedimentation in the Pacific coast basins of western Oregon and California.

In making these evaluations, the measures of sedimentation used were the average annual suspended sediment discharge from watersheds, the seasonal or annual catches of erosion from slopes, or reservoir deposition measurements. Average annual suspended sediment discharge for a watershed was obtained from suspended sediment samples, relating suspended sediment to stream discharge, then obtaining from streamflow discharge frequencies sediment discharge frequencies and finally average annual suspended sediment discharge (table 1). Seasonal or annual catches of erosion from slopes were measured by means of half-round steel troughs placed on contour along slopes or in debris chutes or channels (*6*). Reservoir deposition was determined by measuring changes in reservoir capacity resulting from sediment deposition.

TABLE 1.—*Average annual suspended sediment and discharge from sediment sampling and streamflow duration before and after logging of 1-square-mile of the 4-square-mile Castle Creek, Yuba River headwaters, Calif., 1958 and 1959*

BEFORE LOGGING, 1958

Discharge class	Mean discharge	Streamflow frequency	Relative volume	Sediment samples	Streamflow in various sediment concentration classes, p.p.m.						Average sediment concentration [1]
					< 12.5	13–27	28–72	73–142	143–400	>400	
	C.f.s.	*Percent*		*Number*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*P.p.m.*
< 1	0.2	44.0	0.006	2	44.0	.....	.....	.....	.....	.....	7
1–10	4.1	30.0	.089	15	26.0	4.0	.....	.....	.....	.....	9
11–40	20.2	14.00	.200	14	7.0	6.0	.....	.....	1.0	.....	36
41–100	60.0	8.8	.380	6		7.3	1.5	.....	.....	.....	25
101–200	125.0	2.8	.258	15	.2	.4	.6	1.4	.2	.....	104
201–250	210.0	.25	.038	0	.....	.....	.....	.....	.25	.....	(190)
>250	270.0	.15	.029	6	.....	.....	.....	.....	.....	0.15	(430)
Total	.....	100.00	1.000	58	.....	.....	.....	.....	.....	.....	.....
Average	14.7	.....	.....	.....	7	20	50	108	270	430	[2]64

AFTER LOGGING, 1959

< 1	0.2	44.0	0.006	11	36.0	4.0	4.0	.....	.....	.....	12
1–10	1.0	30.0	.089	13	23.1	.....	.....	4.6	2.3	.....	43
11–40	21.0	14.0	.200	18	7.8	4.7	1.5	.....	.....	.....	16
41–100	63.0	8.8	.380	12	2.2	2.2	.7	2.2	1.5	.....	83
101–200	130.0	2.8	.258	3	.....	.....	.....	.....	1.9	0.9	413
201–250	216.0	.25	.038	.....	.....	.....	.....	.....	.....	.25	(1,700)
>250	280.0	.15	.029	.....	.....	.....	.....	.....	.....	.15	(3,200)
Total	.....	100.00	1.000	57	.....	.....	.....	.....	.....	.....	.....
Average	15.3	.....	.....	.....	7	20	50	108	210	980	[3]303

[1] Values in parentheses were taken from sediment concentration-discharge curve.
[2] 935 tons sediment per year per square mile.
[3] 4,600 tons sediment per year per square mile.

The measures of the meteorologic potential used were storm rainfall intensity and snowmelt characteristics of unit areas, antecedent precipitation, and rain-snow characteristics of the precipitation. In some studies, these variables were used directly; in others, these variables were used to predict two characteristics of the runoff: total volume and peakedness or intensity of runoff.

The topographic potential was assessed from variables of elevation, slope characteristics, and channel patterns and characteristics. Elevation was usually not expressed explicitly, but in terms of how it affected rain-snow distribution of the precipitation, precipitation amounts, and soil formation. Slope characteristics were indexed by the slope of streams of 1-mile mesh length (*4*), thus largely eliminating problems of mapping scale. Channel patterns and characteristics were determined by using Horton's (*10*) procedures.

The sediment associated with the soil potential was determined by taking soil samples under standard conditions and relating their physical characteristics to soil formation factors. The physical erodibility factors — surface-aggregation ratio (*4*) and dispersion ratio (*14*) — were related to rock type, vegetation type, and climatic variables by regression analysis.

The effects of land use and condition on sedimentation were determined for variables by indexing the type and degree of forest land

cutting, forest fires, broad categories of cultivation, extent of road development, length of unstable stream banks, and condition of geologic instability.

Analytical Methods

In studies using many watersheds, we determined the relation of sediment discharge of each to its watershed characteristics by multivariable analysis. Specific variables were chosen to give quantitative expression to the watershed characteristics. A variable had to meet two requirements: (1) It should be related to sediment discharge, and (2) it could be expressed in terms applicable to individual land areas within watersheds. The value of a variable for an entire watershed was obtained by averaging the value of the variables — taken from maps — for the individual land areas within the watershed.

Results of analysis in half a dozen groups of watersheds along the Pacific coast and some recent additional studies of soil erodibility were used to interpret and give quantitative expression to the principal sediment sources and causes in western Oregon and California. In other areas, the results will have quantitative meaning only if climate, topography, and geology are similar, but the variables themselves may be useful in studies of sedimentation in many areas.

The results of analysis of measured sediment production to the various cause-variables are summarized in table 2. Shown are the specific measure of sediment discharge involved, the general analytic model, the specific analytic model, and a definition of variables, together with a regression coefficient giving the independent effect of that variable on sediment discharge. These results were used to interpret and infer the differences in sediment discharge associated with differences in cause-variable.

TABLE 2.—*Regression models and results relating sedimentation to variables of streamflow, topography, soil, and land use and condition potentials*

Models and results	Explanation	
(1) Sedimentation model $SS = f(Sf, T, S, Lu)$	Suspended sediment discharge (tons/sq. mi./yr.) equals function of streamflow (Sf), topography (T), soil (S), and land use (Lu) (4).	
(2) Log SS $= -4.721$	Regression constant.	
$+1.244 \log MAq$	Mean annual runoff, c.f.s./sq. mi.; range, 1.09–7.48.	
$+1.673 \log FQp$	Discharge peakedness, based on a separate regression analysis which related peak discharges from water sheds to: geologic rock types, the area receiving rain during storms, and the area in which snow was ripe during the storms (7); range, 1.98–4.30.	Sf
$+0.116 \log A$	Area of watershed; unit, sq. mi., range 56–7,280.	
$+0.401 \log S$	Slope of stream of 1-mile mesh length after method of Horton (10) (see derivation in section on topographic potential); unit, ft./mi.; mean, 910; range, 210–1,510.	T
$+0.0486\ SC$	Silt and clay; fraction of top 6 inches of soil with particle sizes less than 0.05 mm. in diameter; unit, pct.; mean, 23.0; range, 19.1–32.0.	
$+0.482\ S/A$	Surface aggregation ratio; surface area on soil particles of sand and coarser size (>0.05 mm. diameter) divided by the aggregated silt plus clay, cm.²/mg. pct.; range, 0.26–1.78.	S
$+0.942\ R$	Roads; part of watershed area in roads, percent; mean, 0.30; range, 0.05–0.60.	
$+0.0086\ RC$	Recent cutover; part of watershed area cutover in last 10 years, percent; mean, 6.0; range, 0–30.4.	
$+0.0280\ BC$	Bare cultivation; part of watershed in row crops and small grain, percent; mean, 4; range, 0–22.	Lu
$-0.0036\ OC$	Other cultivation; part of watershed area in cultivation other than bare cultivation, percent; mean, 12; range, 0–48.	
(3) Log e_D $= 2.161$	Sedimentation of reservoir during a single year (and regression constant); unit, acre-ft./sq. mi.; range, 0.3–36.0.	
$+0.071 \log A$	Area of the watershed; unit, sq.mi.; range, 1.5–202.	
$+1.619 \log P_{24}$	Maximum 24-hour precipitation for the watershed during the storm; unit, inch; range, 1.5–20.0.	
$+0.410 \log aP$	21-day precipitation antecedent to the maximum 24-hour for the storm; unit, inch; range, 1.0–23.0.	
$+0.370 \log A_{CH}$	Area of main channel of the watershed; unit, acre/sq.mi.; range, 7–9.7.	
$-1.974 \log C$	Cover density on the watershed; unit, pct.; range, 2.0–72.4.	
(4) Log q $= 1.3383$	Maximum yearly peak discharge; unit, c.f.s./sq.mi.; range, 10–550.	
$+0.060 \log A$	Area of the watershed; unit, sq.mi.; range, 1.5–202.	
$-0.975 \log C$	Cover density on the watershed; unit, pct.; range, 2.0–72.4.	
$+0.863 \log PE_3$	Precipitation effectiveness (see equation 3 above).	
$+0.360 (\log PE_3 \times r_b r_s / r_l$	Defined above and on following page.	
$-0.477 \log r_b r_s / r_l$	Watershed physiography expressed as the ratios, by stream order numbers, of stream numbers, slopes, and lengths, after Horton (10) (r_b, r_s, and r_l were expressed as ratios of larger numbers to smaller numbers); unitless; range, 1.1–4.2.	

TABLE 2.—*Regression models and results relating sedimentation to variables of streamflow, topography, soil, and land use and condition potentials*—Continued

Models and results	Explanation
(5) Log PE_3 = 1.188 log P_{24}	Maximum 24-hour precipitation for the watershed during the storm; unit, inch; range, 1.5–20.0.
+0.516 log aP	21-day precipitation antecedent to the maximum 24-hour for the storm; unit, inch; range, 1.0–23.0.
+0.985 log P_1	Maximum 1-hour precipitation for the watershed during the storm; unit, inch; range, 0.2–2.1.
(6) Log e_D = 1.041	Sedimentation of reservoir during a single year (and regression constant); unit, acre-ft./sq.mi.; range, 0.3–36.0.
+0.866 log q	Maximum yearly peak discharge; unit, c.f.s./sq.mi.; range, 10–550.
+0.370 log Ac_h	Area of main channel of the watershed; unit, acre/sq.mi.; range, 7–9.7.
−1.236 log C	Cover density on the watershed; unit, pct.; range, 2.0–72.4.
(7) Log Es = 3.073	Average suspended sediment content of streamflow in p.p.m.
−2.430 log C	Average cover density on watershed, pct.
+3.427 log DR	Dispersion ratio.
(8) DR = 5.7	Dispersion ratio.
+1.6 Kw	Dissociation constant of water (summed for 12 monthly values of $Kw = 0.2681(1.048F)$, in which F is mean monthly temperature in degrees Fahrenheit.
−0.014 Kw^2	Square of previously defined variable.
(9) S/A = 319	Surface aggregation ratio.
−6.39 P	Average annual precipitation, inches.
−7.52 Kw	(Same as in equation 8).
+2.42 KwP	Summation of 12 monthly products of Kw and P.
+0.0404 $(P)^2$	Square of previously defined variable.
+0.0553 Kw^2	Square of previously defined variable.
−0.00693 $(KwP)^2$	Square of previously defined variable.
(10) Log E = 1.115	Total deposition of sediment in individual reservoir and debris basins as the result of the storm; unit, acre-ft.; range, 1.2–74.0 acre-ft./sq.mi.
+0.619 log A	Area of the watershed; unit, sq. mi.; range, 0.1–202 sq. mi.
+1.688 log P	Maximum 24-hour precipitation falling on the watershed during the storm; unit, inches; range, 5.8–15.1 inches.
+0.191 log B	Barren area on the watershed; unit, acres; range, 0.8–83.0 acre/sq.mi.
+0.255 log F	Old fire area—area of fires multiplied by the number of times burned, for fires occurring more than 15 and less than 60 years before the storm; unit, acres; range, 8–1, 810 acres/sq.mi.
−1.316 log C	Average cover density on the watershed; range, 18.5–88.6 pct.

Meteorologic or Hydrologic Potentials and Their Effects

Differences in sedimentation resulting from the differences in the meteorology of unit areas may be related to either meteorological variables or to hydrologic consequences of the meteorology. We may take the volume of runoff and the intensity of runoff as hydrologic consequences. Differences in the volume of runoff may be rather easily appraised by referring to runoff maps, such as those published by the U.S. Army Corps of Engineers for Oregon and California. Differences in the peakedness or intensity of runoff may be appraised in several ways. For example, we have used the relative frequency of rainfall and snowmelt contributions to peak runoff. These have been determined by elevation zones for western Oregon (*7*) and have been extended to California by Wallis (*16*). Table 3 summarizes the frequency of rain and snowmelt by elevation zones for various latitudes on coastal Oregon and California.

The effects on sedimentation of differences in hydrologic potential were appraised for western Oregon watersheds. Differences in the total volume of runoff between watersheds produced differences in sediment production between watersheds by a factor of 11.7. Differences in the runoff intensity produced differences in sedimentation between watersheds by a factor of 3.6. Watersheds with large contrast in both the volume and peakedness of runoff (Wilson watershed as compared with Big Butte watershed) showed a difference in sediment production of 36 times attributable to the difference in hydrologic potential alone. The actual contrast in measured sediment per unit area was almost 70.

Frequencies for the combinations of the two variables, precipitation intensity and antecedent precipitation, have been worked out for southern California watersheds (*1*). Association between these variables and measured sediment deposition in reservoirs is shown in equation 3, table 2. For the same general storm, on March 1, 1938, differences in the hydrologic potential of two watersheds, Cucumonga and Thompson Creek, caused a 3.4 difference in the sediment production (*8*).

Topographic Potential

Slope of the land is closely related to the slope of the minor channels, according to Horton (*10*). He also presented some relation between

TABLE 3.—*Relative rain area, and snowmelt frequencies for latitudes listed*

RELATIVE RAIN AREA FREQUENCIES[1]

Elevation (feet)	Latitude									
	44°	42°	41°30′	41°	40°30′	40°	39°30′	39°20′	39°	38°30′
0–1,000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1,000–2,000	.96	.99	.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2,000–3,000	.87	.92	.93	.94	.95	.96	.97	.97	.97	.98
3,000–4,000	.75	.81	.83	.84	.86	.87	.88	.88	.89	.90
4,000–5,000	.59	.68	.70	.72	.73	.74	.76	.77	.77	.79
5,000–6,000	.39	.50	.52	.54	.56	.58	.60	.61	.62	.64
6,000–7,000	.17	.29	.32	.34	.36	.38	.41	.42	.43	.45
7,000–8,000	.00	.06	.09	.12	.14	.17	.19	.20	.21	.23

Elevation (feet)	Latitude									
	38°	37°30′	37°	36°30′	36°	35°30′	35°	34°30′	34°12′	34°
0–1,000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1,000–2,000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2,000–3,000	.98	.99	.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00
3,000–4,000	.91	.92	.93	.94	.94	.95	.96	.96	.97	.97
4,000–5,000	.80	.81	.82	.83	.85	.86	.87	.88	.88	.89
5,000–6,000	.65	.67	.69	.70	.72	.73	.75	.76	.77	.77
6,000–7,000	.47	.49	.51	.53	.54	.57	.58	.60	.61	.62
7,000–8,000	.25	.28	.30	.32	.34	.36	.38	.41	.41	.42

RELATIVE SNOWMELT FREQUENCIES[2]

Elevation (feet)	Latitude								
	44°	42°	41°30′	41°	40°30′	40°	39°30′	39′	38°30′
0–1,000	1.00	1.00	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1,000–2,000	.99	.993	.995	.998	1.000	1.000	1.000	1.000	1.000
2,000–3,000	.92	.939	.941	.953	.969	.989	.999	1.000	1.000
3,000–4,000	.82	.840	.846	.866	.889	.921	.967	.991	.999
4,000–5,000	.69	.712	.720	.745	.772	.819	.883	.935	.967
5,000–6,000	.54	.570	.574	.604	.634	.688	.770	.837	.881
6,000–7,000	.38	.410	.416	.449	.481	.539	.631	.711	.768
7,000–8,000	.....	.242	.299	.288	.318	.379	.475	.564	.629

Elevation (feet)	Latitude								
	38°	37°30′	37°	36°30′	36°	35°30′	35°	30°30′	34°
0–1,000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1,000–2,000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2,000–3,000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
3,000–4,000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
4,000–5,000	.983	.992	.997	.998	1.000	1.000	1.000	1.000	1.000
5,000–6,000	.912	.937	.950	.969	.968	.972	.976	.978	.980
6,000–7,000	.807	.840	.860	.873	.885	.893	.900	.904	.908
7,000–8,000	.674	.714	.740	.755	.772	.780	.789	.796	.800

[1] Values taken from a smoothed curve through Anderson and Hobba's (7) relative rain area frequencies for the central Willamette Valley.

[2] Values taken from a smoothed curve through Anderson and Hobba's (7) relative snowmelt frequencies for the central Willamette Valley area.

channel lengths, channel slopes, and channel order number. His studies make possible a valid determination of slope that is independent of map scale. The slope of streams of 1-mile mesh length, for example, can be derived from simultaneous plotting of stream slopes and stream lengths against order number, and reading the slope of a stream whose length is 1-mile (4). To compute sediment discharge, we used the relation of sediment discharge to slope of the streams given by equation 2 in table 2. For a range in slopes from 210 to 1,510 feet per mile, sediment discharge differed by a factor of 2.2.

Channel patterns and characteristics that affect sedimentation include channel areas, which affect the availability of material for transport and channel drainage patterns that affect the hydrologic potential and, hence, affect sediment production. For many watersheds of southern California these effects were summarized by Anderson (1).

The relation of channel area to sediment production, from table 2, equation 3, can be used to compute the effect on sedimentation of differences in channel area. Thus, Tujunga has a sediment potential 98 percent higher than

Big Santa Anita because of channel area alone.

The effects of channel patterns may be trivial for a small storm or years with small discharges, yet of considerable importance in the large events that produce most of the sediment discharge from watersheds. Horton's (*10*) bifurcation, slope, and length ratios have been found to affect the hydrologic potential (*2, 8*). The definition of a composite variable and its relation to discharge is shown in equations 4 and 5 of table 2, and the relation between discharges and sedimentation is shown in equation 6. In a large storm, 50 percent greater sediment production from the San Gabriel watershed is expected than from the Santa Anita watershed because of the differences in the channel pattern alone. But in a small storm, only about 20 percent increase in sediment would be expected (*2*).

Soil Potential

Soils — the primary material of sedimentation — vary in their resistance to erosion. Sediment discharge from watersheds is related to the lack of resistance.

Sediment discharge from watersheds has been related to the dispersion ratio (*14*) and the surface-aggregation ratio (*4*) indexes that measure variation in soil stability. The wide differences in these properties of watershed soils can be determined by relating them to map characteristics, such as geologic type and vegetation type, and to climatic variables, such as temperature and precipitation. Thus, we introduce the submodel: Soil erodibility = function (rock type, vegetation type, climate, and topography), and evaluate this model.

Sediment discharges vary with erodibility in the south coastal watersheds of California (*3*) and the west coastal watersheds of Oregon (*4*). These variations are illustrated in equations 2 and 7, table 2. Sediment production from watersheds of western Oregon differ by a factor of 5 because of variation in the soil erodibility. In the south coastal watersheds even greater variations were found. For example, in the watersheds of the Cuyama badlands, the Miocene Continental deposits had a sediment potential 75 times as great as the marine shales of nearby watersheds.

The variation of these erodibility indexes with the soil-forming factors of rock type, vegetation, and climate (*11*) has progressed through a succession of refinements (*3, 4, 9*).[1] Highly significant and useful relations of soil erodibility to rock type were found. Table 4 summarizes the relation of two erodibility indexes to rock types.

[1] WALLIS, J. R., and WILLEN, D. W. SOME HYDROLOGIC CHARACTERISTICS OF SURFACE SOILS TAKEN FROM CALIFORNIA'S WILDLANDS AND RELATED TO VEGETATION, CLIMATE AND GEOLOGY BY REGRESSION ANALYSIS. Paper presented 2d Western National Meeting, Amer. Geophys. Union, Stanford, Calif., Dec. 28, 1962.

TABLE 4.—*Relation of soil erodibility indexes—dispersion ratio (DR) and surface-aggregation ratio (S/A)—to geologic-rock type*

Rock type	DR[1]	DR[2]	S/A[2]	DR[3]	S/A[3]	DR[4]	S/A[4]
Recent volcanics		60	275				
Young volcanics		56	204				
Acid igneous		54	164	52	120		
Granite and rhyolite						51	149
Quartz diorite						56	132
Grano-diorite						57	112
Diorite						48	71
Cenozoic nonmarine sed.						48	80
Miocene continental	50						
Schist and phylite				40	99	52	76
Metamorphics				49	47	48	67
Cenozoic marine sed.				47	68	40	58
Miocene marine	23	27	26				
Oligocene		28	41				
Eocene	16	30	62				
Quaternary terraces	42						
Upper Cretaceous sed.	37	50	57				
Lower Cretaceous mar.	14						
Basalt and gabbro		34	59	53	51	46	57
Jurassic Triassic		31	53				
Carboniferous volcanic		44	68				
Pre-Cenozoic marine				47	61	47	52
Carboniferous		57	60				
Devonian		23	23				
Peridotite and serpentine				39	42	37	43
Andecite						44	42
Eocene volcanic		25	17				

[1] Reference (*3*).
[2] Reference (*4*).
[3] Reference (*9*).
[4] See Wallis and Willen, footnote 1.

André and Anderson (*9*) concluded that variation in the erodibility of soils develops under different cover types. But now this does not appear to be generally true; when more sensitive climatic variables are included in the analysis, only for a few combinations of rock types and cover types do differences in erodibility occur. Under brush, high erodibility is associated with both marine and nonmarine soft Cenozoic sediments (table 5). Under grass, the granitic rocks display high erodibility, and this also seems to hold for the quartz sericitic schists that are common in the California coast range. Under Douglas-fir forests, the dispersion ratio and — to a lesser degree — the surface-aggregation ratio are low.

TABLE 5.—*Coefficients of interaction of cover type and rock type on surface aggregation ratio, northern California*[1]

Cover type	Rock type											
	Granite and rhyolite	Grano-diorite	Quartz diorite	Diorite	Andecite	Basalt and gabbro	Peridotite and serpentine	Schist. and phylite	Meta-morphic	Cenozoic non-marine sediments	Cenozoic marine sediments	Pre-Cenozoic marine sediments
Fir	.....	+11	+10	−3	.....	+2	+12	−8	−3	.....	.....	+2
Pine	−50	+4	+14	.....	+26	+16	+16	−1	−23	−15	.....	+2
Brush	−10	−21	−18	+12	+7	+11	−11	−14	−10	+32	+54	+3
Grass	+45	+1	0	−12	−33	−18	−7	+29	+19	−9	−16	−4

[1] Surface-aggregation ratio for a rock-type deviate by the amounts shown for the different cover types.

The relation of erodible soil development to climate is of most interest to the soil morphologist, and certainly is of some interest to the sedimentationist. Some schools of soil morphologists would have us believe that climate was the sole control of soil development. We have seen that the rock type is the dominant influence — at least for the young residual soils of the Pacific coast mountains.

But climate is important. Early studies showed differences in soil erodibility with simple climatic segregations of elevation and geographic zone (*9*). More recent studies[2] have shown that a climatic factor can be expressed from the amounts and temperature of the percolating rainfall. The dispersion ratio is related to the dissociation constant of water (equation 9, table 2) ; the surface-aggregation ratio of the soil is related to the amount of precipitation, the dissociation constant of water for that precipitation, and their joint product (equation 9, table 2). Variation in soil erodibility between areas of northern California associated with these climatic variables is by a factor of 1.4.

Soil Supply Factors

Depending on the particular sedimentation product under study, soil characteristics other than erodibility become important in predicting sedimentation. When suspended sediment discharge is being evaluated, the supply of silt and clay in the soil becomes important (equation 2, table 2). When the effect of sediment on water quality in terms of the number of days per year with turbid streamflow (> 27.5 p.p.m.) is being tested, the percentage of coarse gravels in the watershed soil becomes important. Streams in areas of gravelly soils cleared up sooner after storms; hence, had fewer days with turbid flow. Soil texture may cause reservoir deposition to vary even though total sediment discharge is the same; reservoir trap efficiency is greater for watersheds with coarse-textured soils. When we sedimentationists find it desirable to consider separately the effects of the soil on runoff and the effects of runoff on sedimentation. other soil characteristics and their formation will become important.

Land Use and Condition Potential

Wide differences in sediment discharge are associated with differences in land use and condition — both past and present. Factors include forest fires, logging, road building, cultivation, and natural geologic instability.

Logging of forest lands changes the sediment discharge. In Oregon, studies have shown 80 percent greater sediment discharge in one watershed than another, associated only with logging during the previous 10 years (from equation 2, table 2). The first year after one-fourth of a watershed in the Sierra Nevada was logged, sediment discharge increased by a factor of 6 (table 1) ; the second year, the factor fell to 2 (*15*).[3] For the logged area the rates were 17 and 5 times the prelogging rate.

Forest fire effects on sediment discharge have depended on the sequence of storms that followed the fire and the rate that vegetation recovered. For 10 years after a fire that burned

[2] See footnote 1, p. 27.

[3] ANDERSON, H. W. and RICHARDS, L. G. FOURTH PROGRESS REPORT, 1960-1961 COOPERATIVE SNOW MANAGEMENT RESEARCH. U.S. Forest Service, Pacific Southwest Forest and Range Experiment Station, pp. 142-204. 1961. [Processed.]

30 percent of the Santa Ynez watershed, sediment deposition averaged 5,400 tons per square mile per year, 3.3 times the prefire rate (*5*). In major floods, such as the 1938 flood in southern California, sedimentation increased by a factor of 3 for the flood occurring the first year after the burn; only a 16 percent increase was indicated 15 years after the burn (*8*).

Fire may affect erosion differently than it does sediment discharge. The burn in the Arroyo Seco watershed in 1956 produced erosion from watershed slopes in channels 4 to 17 times the prefire rate, yet no sediment discharge reached the downstream Devil's Gate Reservoir in that light runoff year (*12, 13*). Materials eroded from the slopes contribute sediment for years after a fire. Fires 15 to 60 years old still affected sediment discharge in a major flood (*8*). Differences in these old fires between watersheds produced differences in sediment discharge by a factor of 4 during a major storm (equation 10, table 2).

Roads in watersheds produce differences of a factor of 3 in sediment discharge between watersheds in western Oregon (equation 2, table 2). In southern California, roads were a major contributor to sediment discharge during the 1938 flood (*8*). Erosion from road fills for the period 1934-47 averaged from 6 to 10 times that from adjacent nonroad areas; about three-fourths of this eroded material came in the 1938 flood.

Cultivation effects differ markedly between bare cultivation, such as row crops, and other cultivation, such as grain. A difference by a factor of 5 in sediment production is indicated by the analyses in western Oregon (equation 2). That is, 80 percent reduction in sedimentation would be expected from a change from bare to nonbare cultivation.

Geologic unstableness is a major driving force in sedimentation. Where recent uplifts have rejuvenated mountain slopes, major changes in sediment discharge result. Rejuvenated slopes produced as much as four times as much sediment discharge as did similar nonrejuvenated slopes (*6*). When a fire destroyed the vegetation, a larger contrast occurred — the rejuvenated slope produced 11 times as much sediment discharge as that of the nonrejuvenated (*12*).

Application of Results

Differences in the hydrologic, topographic, and land use and condition potentials may be combined to estimate the sediment discharge potential for individual parts of watersheds (*4*). For the Willamette basin in Oregon, for example, a map of the sediment potential was prepared and major sediment-producing areas delineated. Sediment sources were:

Source:	*Sediment discharge (Tons per year)*	*(Percent)*
Forest lands (5,460 square miles)	470,000	24
Agricultural lands (1,120 square miles)	430,000	22
Main channel banks (39 miles)	1,055,000	54
Total	1,955,000	100

By combining the effects of all the various sediment potentials, we have found that the sediment potential for nonagricultural lands varies by a factor of 100 and from agricultural lands by a factor of 7. From these results, areas where caution in management may be needed can be determined and the effectiveness of certain types of management can be estimated.

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RESIDENTIAL CONSTRUCTION AND SEDIMENTATION AT KENSINGTON, MD.

[Paper No. 3]

By H. P. GUY, *hydraulic engineer, U. S. Geological Survey, Fort Collins, Colo.*

Abstract

Sediment transported in storm runoff near Kensington, Md., during the transformation of part of a 58-acre area from rural to residential land use was measured for 25 storm events from July 1959 to January 1962. These data were used with the water discharge record of nearby Rock Creek in a multiple regression analysis to show the magnitude and trend of sediment movement with time.

Total sediment discharged from the area affected by urbanization was 189 tons per acre for the entire period of construction and the subsequent return to a reasonably stable residential area. The high yield of sediment from the Kensington area is attributed to (1) the rolling topography, 3 to 25 percent slope, (2) a very friable soil and subsoil, (3) the construction of a street in the major drainage channel, (4) a tendency for construction methods to expose extensive areas of the soil for a long period of time, and (5) a substantial amount of the 42 inches of annual rainfall occurring at a rate in excess of the infiltration capacity of unprotected soil.

Introduction

Population pressure in and around cities causes much urban construction that is tied to the mantle of soil and subsoil. This includes all the construction for the industrial, commercial, and residential developments and the necessary highways, streets, and utilities. Some of the impact of urban growth on the general water regime was discussed by Savini and Kammerer.[1] The impact of urbanization on sediment accumulation in small reservoirs has been discussed by Guy and Ferguson.[2]

[1] SAVINI, JOHN, and KAMMERER, J. C. URBAN GROWTH AND THE WATER REGIMEN. U.S. Geol. Survey Water-Supply Paper 1591A, 39 pp. 1961.

[2] GUY, H. P., and FERGUSON, G. E. SEDIMENT IN SMALL RESERVOIRS DUE TO URBANIZATION. Amer. Soc. Civil Engin., Jour. Hydr. Div. 88 (HY2): 27-37. 1962.

The population growth of our cities and the pressure to occupy extensive land areas around them are mostly attributed to advances in agricultural technology and private transportation. Thus, the relatively high economic status of the urban population and relatively cheap transportation have combined to "price" the farmer off the land adjacent to cities. In many instances the use of more single- instead of multiple-family dwellings, larger parking lots, and one-level school and commercial buildings causes an increase of urban land use per capita.

This paper gives an example of the effects of residential construction on the sediment transported from a 58-acre drainage area at Kensington, Md. — a part of the metropolitan area of Washington, D.C. The duration of the study is from July 1957 to April 1962.

The soil at Kensington contains a considerable amount of sand and silt. The average size distribution of two subsoil samples in the basin is 14 percent clay, 30 percent silt, 43 percent sand, and 13 percent gravel. Thus, the soil is very friable when acted upon by the erosive forces of raindrops and by the surface runoff flowing in sheets, rills, and larger channels. Under rural or natural conditions, the soil is usually well protected by vegetative cover.

The slope of the land surface in the construction area ranges from 3 to 10 percent. Artificial slopes may be much steeper. The layout of streets in the basin is irregular. One of these streets is in the principal drainageway in the upper part of the basin.

The average annual precipitation on the area is about 42 inches. The average amount of stream runoff is about 16 inches, of which about 4 inches is overland or surface runoff. Areas with little or no vegetative cover usually have a higher portion of the precipitation leaving the basin as surface runoff.

Basic Measurements

The determination of the amount of sediment

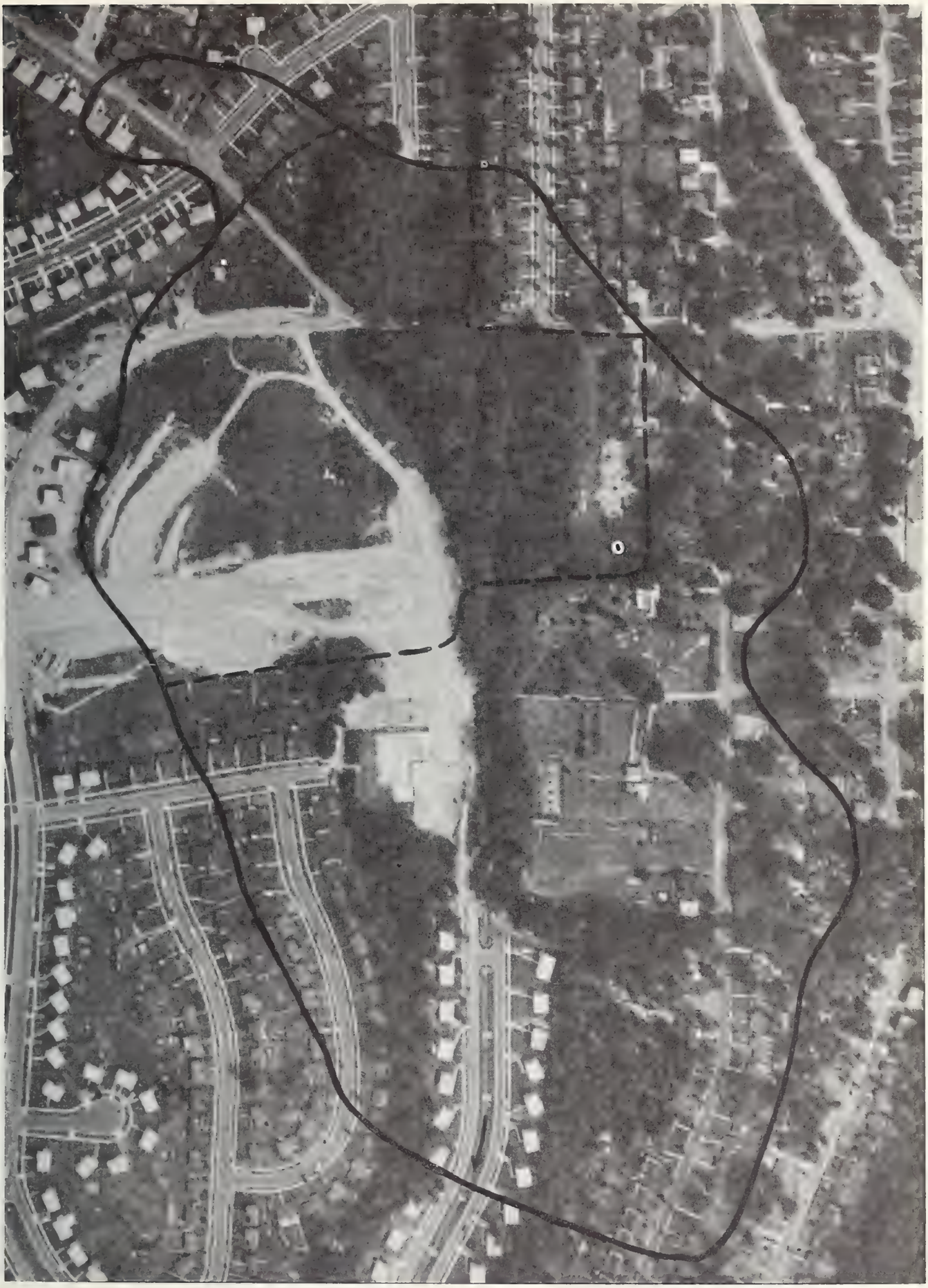

FIGURE 1. — Photograph taken August 28, 1957, on which the construction area in the basin is outlined. Scale, 303 feet per inch.

FIGURE 2. — Photograph taken May 7, 1961, on which the completed residential construction is shown. Stabilization of the construction area and the drainage channels caused a diminution of sediment outflow for many months. Scale, 317 feet per inch.

transported from the basin was accomplished by use of a combination of actual observations and a system of computation through correlations. The sampling was done on only selected storm events, because the small size of the basin caused the surface runoff from the construction area to drain from the basin within a few minutes of the rain. Generally, several samples were obtained for the selected storms in order to define the variation of sediment concentration in transport. The samples were velocity-weighted over the cross section of flow moving through a drop spillway, and therefore, except for particles too large to enter the nozzle of the sampler, represent the total sediment discharge.

Sediment concentration and water discharge data were collected for 25 storm events plus some miscellaneous single observations during other storm periods. The first observations were made in July 1959 about the time of maximum construction in the basin. The sediment transport determinations for the unmeasured storms was accomplished through correlation with water discharge of nearby Rock Creek on which a continuous record is available.

The water discharge for the storms having sediment measurements was computed from hydrographs drawn from instantaneous observations of flow depth in the notch of the drop spillway. The stage-discharge relation was established by measurements with a pigmy current meter. The unmeasured water discharge from the basin was determined by correlation with water discharge from nearby Rock Creek. All correlative relations for both sediment and water were made on the storm event basis.

A precise record of the environmental conditions of the basin during the period of record was not attempted, because of the effect of the varying stage of construction of a group of houses on the amount of sediment yield for a given storm characteristic and given season of the year. It is logical to assume that the maximum sediment yield would occur when the maximum 10 acres of construction was exposed. The total area exposed during the period of record can, however, be measured quite precisely from aerial photographs. (See figs. 1 and 2.) The total construction was for 89 single dwelling houses on 20½ acres.

Results for a Single Storm

The water discharge hydrograph and sediment concentration curves are shown in figure 3 for one of the larger storms (August 4, 1960). The streamflow hydrograph was drawn on the basis of four observations of water stage and the stage-discharge curve. The discharge near the peak rate of flow was checked by observations of float velocity moving through an 80-foot reach of the channel immediately upstream from the drop spillway.

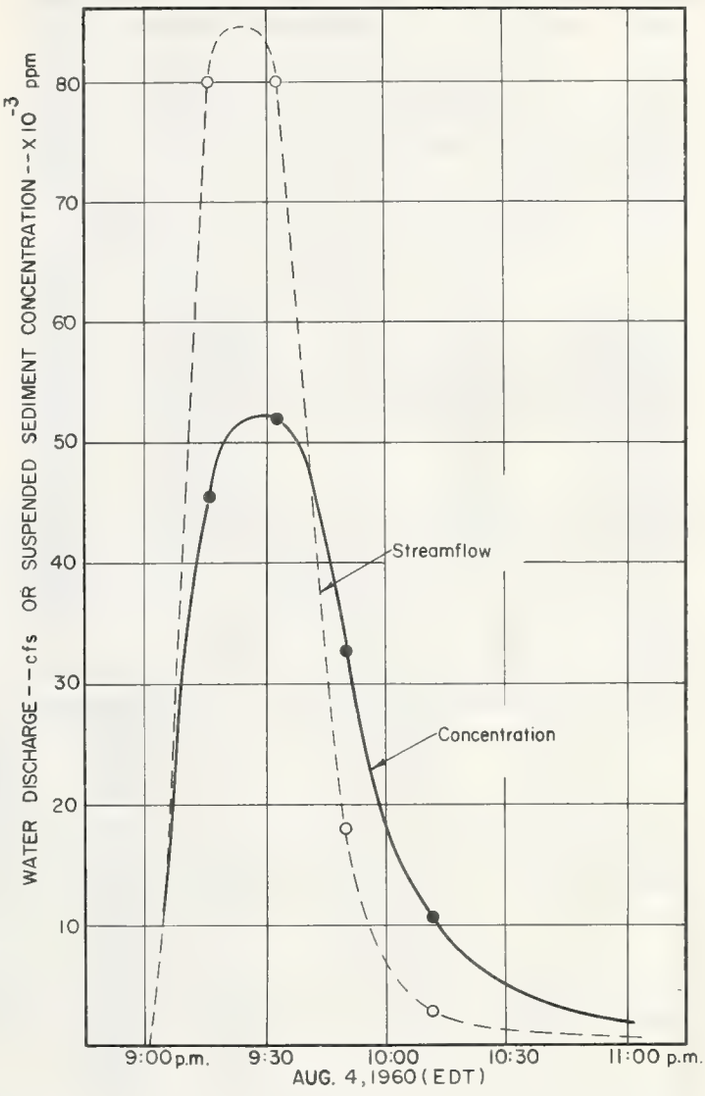

FIGURE 3. — Runoff hydrograph and suspended-sediment concentration graph for storm of August 4, 1960, on a 58-acre area at Kensington, Md.

Four depth-integrated suspended-sediment samples were obtained at the time of the water discharge measurements. A concentration graph was drawn on the basis of these data and the concentration data for other storms. The relative amount of sand in each concentration sample was determined by wet sieving the sediment. The percentage of sand in each consecutive sample during the storm is 67, 80, 77, and 44. The amount of coarse sediment as contrasted with the amount of fine sediment in transport is considered indicative of the relative amount of sediment derived from channel erosion as contrasted with sheet erosion.

The hydrograph and concentration graphs were subdivided as shown in table 1 for computation of water and sediment discharge. The

table, the graphs, and the basic data provide information for summarizing the total and the peak intensity of rainfall, streamflow, and sediment as shown in table 2. This table allows more meaningful comparison of this storm with other storms and with other drainage areas.

TABLE 1.—*Subdivision and computation of water and sediment discharge for storm the evening of Aug. 4, 1960*

Time (p.m.)	Interval	Runoff discharge	Sediment	
			Concentration	Discharge
	Hour	*C.f.s.*	*P.p.m.*	*Tons*
9:00–9:06..	1/10	6.5	7,000	0.5
9:06–9:12..	1/10	39	28,000	12.3
9:12–9:18..	1/10	74	43,000	37.1
9:18–9:30..	1/5	84	51,200	100.3
9:30–9:36..	1/10	79	51,700	47.7
9:36–9:42..	1/10	56	49,000	32.0
9:42–9:48..	1/10	34	41,500	16.5
9:48–10:00.	1/5	13	27,000	7.9
10:00–10:12.	1/5	4.7	14,400	1.5
10:12–10:42.	1/2	1.7	6,000	.5
Total...	—	[1]50.04	—	256.3

[1] In c.f.s. = hours.

TABLE 2.—*Summary of rainfall, streamflow, and sediment discharge, both total and peak intensity, for storm of Aug. 4, 1960*

Storm items	Total	Peak rates for maximum 12-minute period
Rainfall....	1.82 inches from 9 to 11:20 p.m. 1.68 inches from 9 to 9:33 p.m.	Estimated 4 inches per hour.
Streamflow..	2.1 c.f.s.-days 0.87 inch.	84 c.f.s. (from 58 acres) 927 c.f.s. per sq. mi. 1.44 inches per hour.
Sediment...	260 tons.	51,200 parts per million. 8.4 tons per minute for 58 acres. 12,000 tons per day for 58 acres. 132,000 tons per day per sq. mi.

Analysis of Data

The quantity of sediment discharged from the basin during the period of construction was computed by the following generalized steps:

(1) The amount of water and the sediment discharged for each of the 25 sampled storm

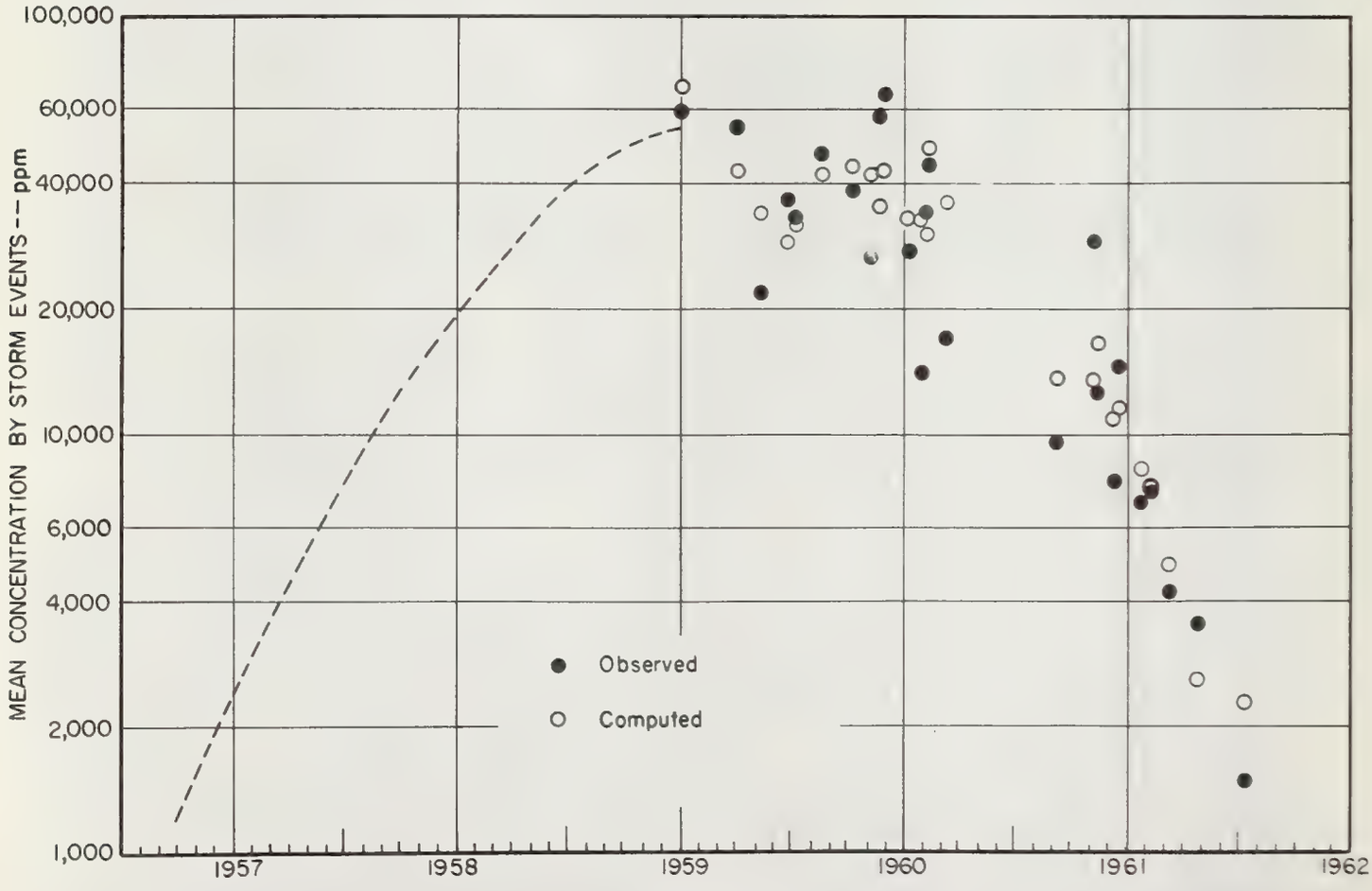

FIGURE 4. — Variation of mean sediment concentration of storm runoff from an area of residential construction at Kensington, Md., 1957-62. The line to July 1959 is estimated on basis of visual observations of construction area and the drainage channels.

events were computed from sketches of the hydrographs and sediment concentration graphs. Figure 3 and table 1 provide an example for one of the 25 storms.

(2) The second step consists of listing variables associated with the storm-to-storm variation of sediment concentration or discharge. Due to the drastic changes of environment in the drainage basin, the effects of which cannot be evaluated numerically, a measure of time (M_t) from the beginning of the record is essential (fig. 4).

Some of the remaining storm-to-storm variation can be related to (a) the magnitude of the storm measured in terms of runoff quantity (Q_w) or rainfall quantity (R_q), (b) the season or time of year which can be evaluated in terms of the mean air temperature (T_a), (c) the storm intensity determined from the relative peakedness of the hydrograph (P_n), and (d) the antecedent condition of the basin measured in terms of the base flow in nearby Rock Creek.[3] Tables 3 and 4 summarize the data for these variables for each of the 25 storms for the construction basin and for the Rock Creek basin (values with subscript r), respectively.

(3) Graphical correlation was used in the third general step to determine the necessary transformation of the variables to yield linear relationships. The values of T_a can be used directly. The function (f) for transformation of M_t is

$$f(M_t) = \log\left(\frac{M_t - 5}{1.5}\right)^{\left(1 + \frac{Mt-5}{10}\right)}$$

All other variables are transformed with log base 10. The values for Q_w are multiplied by 100 and the values for R_q and R_{qr} are multiplied by 10.

TABLE 3.—*Hydrologic and sedimentologic data by storm events from a drainage area affected by residential construction*

Date	Precipitation (R_q)	Water discharge (Q_w)		Sediment		Peak flow (P_n)	Mean air temperature (T_a)
		C.f.s.-days	Inches	Discharge (Q_s)	Concentration (C)		
1959	*Inches*			*Tons*	*P.p.m.*	*C.f.s.*	*°F.*
July 1	1.7	0.81	0.332	128	59,200	25	74
Oct. 1	.9	.39	.160	60	54,500	9.0	61
Nov. 7	.8	.25	.103	14.7	21,800	1.8	48
Dec. 28	.44	.048	.020	5.0	37,200	2.0	35
1960							
Jan. 3	.8	.156	.064	14.3	32,800	5.0	33
Feb. 18	.7	.59	.242	73	46,700	11.1	34
Apr. 3	.4	.194	.080	20.3	38,700	1.6	49
May 8	1.5	.76	.312	55	26,800	6.0	61
May 21	.6	.23	.094	37	57,300	12	64
May 22	.6	.39	.160	72	65,300	15	64
July 11	.7	.158	.064	11.7	27,400	5.0	75
July 30	1.0	.53	.218	20.2	14,000	3.1	75
Aug. 3	.8	.52	.213	48	34,000	13	74
Aug. 4	1.8	2.08	.854	256	44,000	84	74
Sept. 12	4.1	2.68	1.102	123	17,000	20	68
1961							
Mar. 8	.4	.233	.096	6.0	9,600	7.8	41
May 7	.6	.149	.061	11.7	29,000	13.5	61
May 12	.3	.37	.152	12.6	12,700	9.0	61
June 9	.22	.032	.013	.7	7,700	2.1	69
June 14	1.0	.48	.197	18.9	14,600	22	70
July 24	.30	.042	.017	.8	6,900	3.3	75
Aug. 9	.6	.21	.086	4.1	7,350	12.9	74
Sept. 3	.30	.125	.051	1.4	4,200	12.3	70
Oct. 21	.5	.45	.185	4.4	3,550	3.4	54
1962							
Jan. 6	.6	.23	.094	.9	1,490	3.9	33

(4) Multiple regression analysis by computer was then used to obtain equations having a minimum standard error of estimate for predicting log C and log Q_w. These are:

1. Log $C = 4.935 - 0.506\ f(M_t) + 0.317 \log P_n - 0.294 \log 10\ R_q$ (Standard error = 0.141 log units.)
2. Log $C = 3.741 + 0.109 \log Q_{wr} - 0.429\ f(M_t) + 0.0062\ T_a + 0.325 \log Q_{br}$ (Standard error = 0.172 log units.)
3. Log $100\ Q_w = -0.579 + 0.896 \log Q_{wr} + 0.0103\ T_a - 0.345 \log Q_{br}$ (Standard error = 0.271 log units.)

The first equation is mostly of academic interest because water discharge and precipitation data were collected for only 25 storms during the period of observation; however, it does

[3] GUY, H. P. AN ANALYSIS OF SOME STORM-PERIOD VARIABLES AFFECTING STREAM SEDIMENT TRANSPORT. U.S. Geol. Survey Prof. Paper 462-E, 46 pp. 1964.

TABLE 4.—*Hydrologic data for Rock Creek for storm events listed in table 3*

Date	Precipitation (R_{qr})	Water discharge (Q_{wr})		Base flow (Q_{br})	Peak flow (P_{nr})
		C.f.s.-days	Inches		
1959	*Inches*			*C.f.s.*	*C.f.s.*
July 1	0.8	170	0.102	18	840
Oct. 1	1.0	135	.081	21	396
Nov. 7	.6	55	.033	33	164
Dec. 28	.6	65	.039	49	121
1960					
Jan. 3	.8	205	.123	41	443
Feb. 18	1.3	895	.536	75	1,080
Apr. 3	1.2	286	.171	110	330
May 8	1.9	323	.193	36	418
May 21	.8	128	.077	35	131
May 22	.5	175	.101	53	401
July 11	1.5	166	.099	19	386
July 30	1.5	171	.102	22	450
Aug. 3	.9	111	.066	18	890
Aug. 4	1.6	522	.312	30	1,750
Sept. 12	3.2	791	.473	28	1,200
1961					
Mar. 8	.3	124	.074	76	99
May 7	.5	160	.096	60	374
May 12	.4	215	.129	57	325
June 9	.2	35	.021	43	283
June 14	.5	148	.089	35	650
July 24	.6	34	.020	34	90
Aug. 9	.4	50	.030	21	194
Sept. 3	.2	12	.007	18	48
Oct. 21	1.8	268	.160	6	531
1962					
Jan. 6	.8	360	.216	28	480

give a basis for evaluating the effectiveness of the other equations when used for determining sediment discharge of the unsampled storms.

From the time of the first observation in July 1959 to the end of the record in April 1962, the continuous recording from the Rock Creek gaging station shows a total of 124 storm events that could result in at least 0.011 inch of runoff or at least 0.2 ton of sediment. The sediment discharged from the drainage basin for each of the 124 storm events was computed by

$Q_s = 0.0027\ C\ Q_w$ when $C < 32{,}000$ p.p.m.

and

$Q_s = 0.0028\ C\ Q_w$ when $C > 32{,}000$ p.p.m.

where C and Q_w are computed from the multiple regression equations 2 and 3. The computed results for C are presented for comparison with the measured values of C in figure 4 for the 25 storms for which data were obtained.

From July 1957, the beginning of construction, to the observation of the first storm event in July 1959, the data from Rock Creek show an additional 63 storm events important to the determination of the sediment contribution from the construction area. The sediment discharged for these events was computed by use of the same formulas for Q_s, the third regression equation for Q_w, and the use of an estimated value for C from figure 4. During the period of record for these 63 events, the value for Q_w was decreased by a factor to compensate for the trend of increasing storm runoff as the 20-acre construction area was cleared of it natural vegetative cover. The factor was varied with time from 0.62 in July 1957 to 1.00 in July 1959. During the sampling period from July 1959 to April 1962 no significant change in the relation of runoff in the test basin Q_w to runoff in Rock Creek Q_{wr} took place.

The computations show that 4,000 tons of sediment in 22 inches of runoff was discharged from the 58-acre basin during the 4¾-year construction period from July 1957 to April 1962. The actual construction area on which 89 single family dwellings were built is 20½ acres. The remaining 37½ acres is mostly a residential area, which was completed prior to this study, and some undeveloped area. A liberal estimate of sediment discharge for the 37½-acre part of the basin would be three-quarter ton per acre per year, or a total of about 130 tons. The net is then 3,880 tons from the construction area, which is equivalent to 189 tons per acre, or 121,000 tons per square mile.

Figure 5 shows the trend of cumulative sedi-

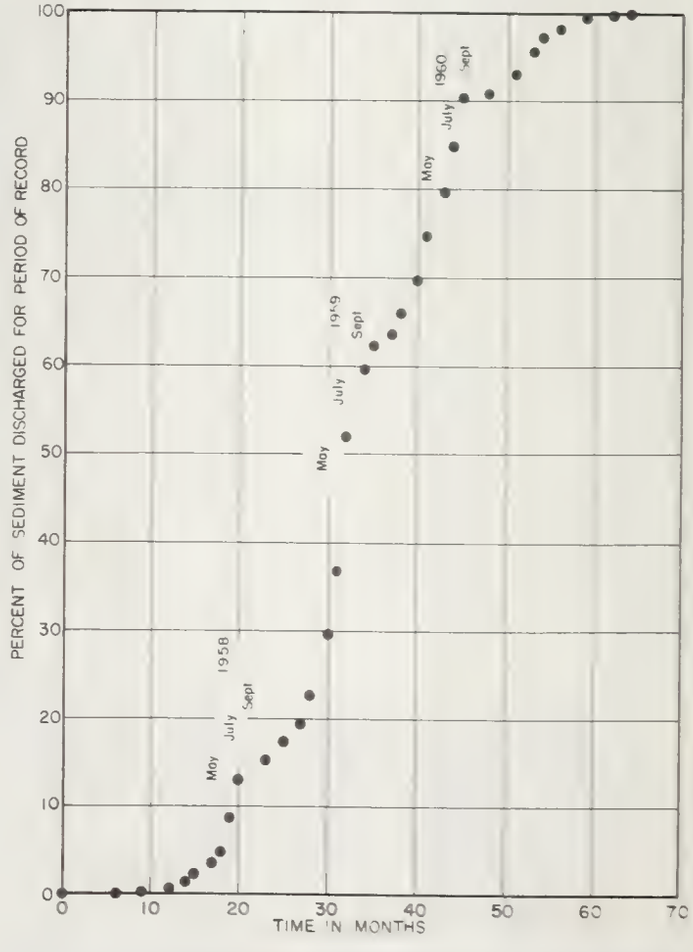

FIGURE 5. — Cumulative sediment discharged from the construction area with time. Note the higher rates of yield during summer months.

ment discharge from the area during the construction period. The rate of sediment discharge yield was more during the summer months than during the remaining months of the year. The maximum year of accumulation was during 1959 when the average concentration of sediment in the runoff was a maximum (fig. 4).

Discussion and Summary

Most of the construction in the 58-acre basin was located on or near the upper part of the basin. As noted in the introduction, one of the streets was built over the principal drainageway. The construction followed a pattern of development in subareas ranging in size sufficient for 5 to 20 houses with as many as 3 subareas exposed at one time. The lack of vegetative cover reduces the infiltration rate and hence causes an increase in surface runoff. Some of the subareas were exposed for as long as 2 years while others were exposed for only about 8 months. The relatively great areas of exposed slopes and channels, therefore, cause intensive sheet, rill, and channel erosion of sediments.

In the early stages of construction in 1957 and 1958, much of the sand eroded from the construction areas was deposited in the downstream channels, giving them the appearance of a true sand-bed stream. In the last year of the record, most of the sand was transported from the channels. Thus, the channels returned to the more natural state with rock and gravel armoring of the bed and with heavy vegetative growth on the banks.

The 189 tons per acre, or 121,000 tons per square mile, determined from the Kensington area may be considerably greater than may be expected for average urbanization around Washington. Measurements of sediment accumulated in Lake Barcroft, also a part of the metropolitan Washington, near Fairfax, Va., between 1938 and 1957 show an average of about 25,000 tons for each of the 9½ square miles urbanized. The Lake Barcroft yield may be low, since much of the housing development, at least in the vicinity of the lake, is "custom built," that is, only the small area for a single home is exposed at a given time and each area has little likelihood of being connected by direct channel to the natural surface drainage system. Also, in the Barcroft area, some sediment is stored in the channels and on the flood plains that drain the area to the lake.

It is reasonable to assume that the streams draining the area around metropolitan Washington will transport 20 million tons of sediment to the Potomac River in the next 20 years given the following:

1. That the population increases by 2 million.
2. That the present population density of 4,000 per square mile will continue.
3. That 500 square miles of rural area will therefore be urbanized.
4. That the sediment discharged to the streams will be 40,000 tons per square mile, or about one-third that measured at Kensington and yet more than that measured at Lake Barcroft.

The results of urbanization under these conditions amount to the movement by streams of 10 tons of sediment for each person added to the city. Such intense movement causes the streams to be muddy part of the time and may cause considerable stream aggradation. The sediment will be deposited in small reservoirs in some cases, but mostly in the navigation and recreation area of the Potomac estuary.

FACTORS RELATED TO GULLY GROWTH IN THE DEEP LOESS AREA OF WESTERN IOWA

[Paper No. 4]

By Craig E. Beer, *assistant professor*, and Howard P. Johnson, *professor, Agricultural Engineering Department, Iowa State University, Ames*

Introduction

Soil erosion became a serious problem in America soon after the beginning of modern agriculture. One type of erosion was the formation of gullies. In 1939, Bennett (*2*) stated there were more than 200 million active gullies in the United States. Since 1939 more gullies have developed. Although progress has been made in the control of gullies, the process of gullying has not been defined by quantitative relationships. In many cases today, the quantitative prediction of gully development is needed for the cost-benefit analysis of structural measures in the Public Law 566 Watershed Program. Since little has been done in research to aid the geologists and engineers to improve their estimates, gross errors may be made in prediction of gully growth. The need for further research is emphasized by a quote from Lueder (*4*):

> It is rather unfortunate that no quantitative observational data, prepared upon a comparative base, have yet been amassed regarding the relationships among length, texture, erodibility, age and hydrology. It is possible

that such data would prove so complex as to defy other than general analysis. Even general analysis would be of value, however.

The objective of this paper is to present part of the results of a study [1] that was designed to meet the following objectives.

1. To select active gullies in the loess soil area and determine, as accurately as possible, the amount of gully growth for a given period in the past by the use of land surveys and aerial photography with ground control.
2. To relate the gully growth in the given period to hydrologic, gully geometry, and watershed variables.
3. To define relationships between factors that are associated with the geometry of the gully.

Investigation

Selection of Gully Study Area

The area used for the gully study was Steer Creek Watershed located in Harrison County, Iowa. This watershed covers approximately 14 square miles. The main gully extends upstream for 10 miles and has many laterals and sublaterals that are active gullies. The gullies in Steer Creek Watershed were well suited for the study and met the following criteria for the selection of a study site.

1. Data must be available to determine accurately the gully development for a 20-year period. It would be desirable to divide the 20-year period into two or three intervals that would increase the number of samples.
2. The land treatment and watershed cover must be available for a 20-year period.
3. Recording rain gage records should be available from a station located near the gullies to be studied.

Methods of Obtaining Data

Aerial flights for 1938, 1949, and 1961 were obtained for Steer Creek Watershed. Contact prints of the 1938 and 1949 flights were obtained from the U.S. Department of Agriculture Commodity Stabilization Service. The 1961 flight was contracted by the Iowa Agricultural and Home Economics Experiment Station. Before the 1961 flight, targets were placed on the ground at right angles to the flight line and were visible on the photographs. The distance between the targets was chained, which made it possible to determine the exact scale of any photograph along the flight lines. In all cases where aerial photographs were available, diapositives of each negative were made and used in a Kelsh Plotter to obtain a topographic map with four- to five-fold enlargement. Thus, it was possible to delineate on the topographic map the gully outlines, land treatment measures, land use, subwatershed outlines, and the natural drainageways from the gully overfalls to the subwatershed divides. The map scale for the 1961 flight was 1 inch equal to 200 feet and, for the 1938 and 1949 flight, 1 inch equal to 500 feet.

A ground survey of Steer Creek Watershed was made by Soil Conservation Service personnel in 1942. This survey provided means for determining gully outlines, gully cross sections, gully profiles, and land use that existed in 1942. The scale of the 1942 topographic map was 1 inch equal to 200 feet. Thus, it was possible to obtain the 1942 and 1961 data from the same map scale.

The choice of weather stations with appropriate precipitation records was limited. The nearest station to Steer Creek Watershed that had recording rain gage records prior to 1938 was the airport at Omaha, Nebr. Therefore, the precipitation records from Omaha were used in this gully study.

Interviews were conducted by the authors with farmers who had been residents in the Steer Creek Watershed during the period 1900-30. From these interviews, it was possible to correlate landmarks with stages of gully development in some areas during this period. Further information on the stage of development of the drainage system in Steer Creek was obtained from the original land surveyors' notes (*1*).

During a 20-year period, many changes, either natural or induced by man, can occur that render some gully growth data unusable. Many cases were encountered in this study where the gully growth had been altered by roads, by installation of land treatment measures, and by excessive timber growth in the gully. It is also extremely difficult to determine the magnitude of some variables related to gullying. In most cases, no continuous record of either land cover or volume of runoff is available. Thus, the validity of any quantitative relationship for gullying is dependent on the quality of the available data.

Analyses

The records (*1*) of 1851 showed no major gully development in Steer Creek. There were three developed laterals averaging 3 feet in width. Information obtained from interviews showed active gullying around 1900 in the laterals that were shown on the 1852 map. However, the development of the remaining 26 major lateral gullies has occurred from 1915 to 1963. The maximum width of the main gully in 1852 was 7 feet at the outlet. As late as 1932, the depth and width of the main was such that it could be crossed easily by livestock and horse-drawn vehicles at many locations. The depth

[1] BEER, C. E. RELATIONSHIP OF FACTORS CONTRIBUTING TO GULLY DEVELOPMENT IN LOESS SOILS OF WESTERN IOWA. 1962. [Unpublished Ph.D. thesis. On file, Iowa State University Library, Ames.]

of the main gully today exceeds 30 feet and the width 50 feet at most locations.

The major part of this gully study was devoted to relating the growth of lateral gullies to hydrologic, gully geometry, and topographic variables. The hydrologic variables that were evaluated for the period of gully growth were precipitation depths and an index of surface runoff. An equation derived by Gray [2] that predicts volume of runoff on a per storm basis was used to evaluate the total runoff for a given gully growth period. The equation expresses the volume of runoff as a function of depth and intensity of precipitation, percentage of watershed area in meadow, and the antecedent precipitation index.

The gully geometry and topographic variables were evaluated from the topographic maps. The geometry variables included the length of the gully from the outlet to the overfall and the surface area of the gully contained within the gully outline on the maps. The topographic variables were evaluated for each gully and included the watershed area and the length along the drainageway from the outlet to the watershed divide.

Regression Analysis With Linear Model for Change in Gully Area

A regression analysis was made on the data of lateral gully development from 1938 to 1961. The data were from 61 samples. One gully could provide three samples by dividing the period from 1938 to 1961 into three intervals. All gully samples had one overfall and were continuous from the outlet to the overfall.

Since regression analysis may be defined as the estimation or prediction of the value of one variable from the values of other given variables, the practical application presents a number of problems. First, there are the problems of estimating the constants of a regression when the form of the relationship is given and the testing of the concordance of some preassigned regression relation with the data. There is also the question of which variables should be included in the relationship. Since the functional relationship or model for predicting gully growth and the variables that should be included were not known, a model that expressed gully growth as a linear function of the variables was assumed for the preliminary analysis.

Table 1 includes five equations derived by using the linear model. These equations were obtained by programming the data for the IBM 650 computer. Five combinations of variables represented by equations 4, 5, 6, 7, and 8 in table 1 were investigated.

Statistically, the equations in table 1 fit the data reasonably well. The R^2 statistic, which according to Snedecor (*5*) measures the fraction or percent of total deviation that is attributed to regression, is 0.70, 0.89, 0.73, 0.89, and 0.89 respectively for equations 4, 5, 6, 7, and 8. Each regression coefficient of the variables in equations 4, 5, 6, 7, and 8 was tested to determine if the value was significantly different from zero. This test of significance is based on the *t*-distribution, and, in a given equation, one considers a regression coefficient to be tested independently of the remaining coefficients. Those coefficients that were significant at the 95 percent level or greater are shown in table 1 by boldface figures. The failure of a regression coefficient to be significant does not necessarily mean that the associated variable should be omitted from the equation. Yates, as quoted by Williams (*6, p. 5*), has the following to say about tests of significance:

TABLE 1.—*Regression equations to predict change in gully area using a linear model* [1]

No.	Equations[2]
4....	$X_1 = -0.906 + \mathbf{0.0022}X_2 - \mathbf{0.0484}X_3 + 0.0098X_4 + 0.0308X_5 - 0.0271X_6 + \mathbf{5.5209}X_7$
5....	$X_1 = -0.324 - 0.0006X_2 - \mathbf{0.0435}X_3 + 0.0063X_4 + 0.0453X_5 - \mathbf{0.0355}X_6 + 0.0013X_8 - 0.00008X_9$
6....	$X_1 = -1.665 - 0.0019X_2 - \mathbf{0.0495}X_3 - 0.0143X_4 + 0.0431X_5 - 0.0814X_6 + \mathbf{5.5576}X_7 + \mathbf{0.0003}X_9$
7....	$X_1 = -0.526 - 0.0017X_2 - \mathbf{0.0465}X_3 + 0.0069X_4 + \mathbf{0.0533}X_5 - 0.0506X_6 - 0.2172X_7 + \mathbf{0.0013}X_8$
8....	$X_1 = -0.240 - 0.0428X_3 + \mathbf{0.0057}X_4 + 0.0443X_5 - 0.0286X_6 + \mathbf{0.0012}X_9 - \mathbf{0.0013}X_{14}$

[1] Coefficients in **boldface** in this table indicate a significant level of 95 percent or greater.

[2] X^1 = Change in gully surface area (acres);
X_2 = Watershed area (acres);
X_3 = Deviation of precipitation from normal (inches);
X_4 = Index of surface runoff (inches);
X_5 = Length of period (year);
X_6 = Terraced area of watershed (acres);
X_7 = Ratio of gully length (L_1) at beginning of period to total length (L) from outlet to watershed divide;
X_8 = Gully length (L_1) at beginning of period (feet);
X_9 = Total length (L) from outlet to watershed divide (feet);
X_{14} = Length from end of gully to watershed divide (feet).

[2] GRAY, D. M., and JOHNSON, H. P. RAINFALL AND RUNOFF RELATIONSHIPS FOR LOESS SOILS OF WESTERN IOWA. Iowa State Univ. Dept. Agr. Engin., Ames. 1961. [Unpublished paper presented at Amer. Soc. Agr. Engin. Meeting, Ames.]

The emphasis on tests of significance, and the consideration of the results of each experiment in isolation, have had the unfortunate consequence that scientific workers have often regarded the execution of a test of significance on an experiment as the ultimate objective. Results are significant or not and that is the end of it. Research workers, therefore, have to accustom themselves to the fact that in many branches of research the really critical experiment is rare, and that it is frequently necessary to combine the results of numbers of experiments dealing with the same issue in order to form a satisfactory picture of the true situation.

On the basis of the preceding discussion, there is no evidence to reject any of the equations in table 1. However, a further check on the validity of equations 4, 5, 6, 7, and 8 was made by substituting the original data into the equations and examining the predicted value of change in gully surface area. All equations gave some predicted values that were negative or less than zero. This result is not desirable and limits the usefulness of the linear model equations. Also, computations from the linear analysis revealed correlations of 0.80 to 0.92 between the following variables:

1. Watershed area and watershed length;
2. Length of period and index of surface runoff; and
3. Length of period and deviations of precipitation from normal.

Regression Analysis With Logarithmic Model for Change in Gully Area

To avoid the problem of negative-predicted values and reduce the interdependence of variables, a logarithmic model with different variable combinations was tried. This model forced the curve through the origin, and no negative predicted values would result from the use of the equation. In the logarithmic model, the logarithm of the predicted variable equals the logarithm of a constant plus the sum of the products of the coefficients times the logarithms of the respective variables. Since the variable X_3, which represented the deviation of precipitation from normal, could be either positive or negative, it was not possible to include this variable in the logarithmic form; the product of X_3 and its coefficient were added to the logarithmic terms. Thus, after taking the antilogarithm of both sides of the model, the equation is of the form represented by equations 9, 10, and 11 in table 2.

TABLE 2.—*Regression equations to predict change in gully area using a logarithmic model* [1]

No.	Equations[1]
9...	$X_1 = 0.013\ X_2^{0.0790}\ X_5^{\mathbf{1.314}} X_6^{-0.0708}\ X_{10}^{\mathbf{0.500}}\ e^{-\mathbf{0.0783}X_3}$
10...	$X_1 = 0.01\ X_4^{0.0982}\ X_6^{-0.0440}\ X_8^{\mathbf{0.7954}}\ X_{14}^{-0.2473}\ e^{-0.0360X_3}$
11...	$X_1 = 0.549\ X_2^{-0.1314}\ X_4^{0.0411}\ X_6^{-0.0575}\ X_{10}^{\mathbf{0.6775}}\ e^{-0.0304X_3}$

[1] **Boldface** coefficients in this table indicate a significance level of 95 percent or greater.
[2] X_1 = Change in gully surface area (acres);
X_2 = Watershed area (acres);
X_3 = Deviation of precipitation from normal (inches);
X_4 = Index of surface runoff (inches);
X_5 = Length of period (year);
X_6 = Terraced area of watershed (acres);
X_8 = Gully length (L_1) at beginning of period (feet);
X_{10} = Gully surface area at beginning of period (acres);
X_{14} = Length from end of gully to watershed divide (feet);
e = 2.71828 (base of natural logarithm).

With the information gained from the linear analysis, the combinations of variables for the logarithmic model were chosen to satisfy the following conditions:

1. One variable would be used to measure the watershed area contributing runoff at the overfall;
2. One variable would be used to measure the length along the gully where growth in surface area results from increased width in the present length of the gully;
3. A lesser number of variables would be used to measure the hydrologic and period-of-time factors; and
4. The remaining variables would be the same as in the linear analysis.

Although the R^2 values for equations 9, 10, and 11 are lower than for the linear model equations, the sign of most of the exponents is the same as if the sign had been determined by reasoning. The watershed area variable becomes positive in equation 9 with the omission of the watershed length variable. However, the sign of the watershed area variable is reversed in equation 11. In equation 9 the period of time (X_5) was used instead of the runoff index (X_4). With a high correlation between X_4 and X_5, the change in sign for the watershed area would not be expected. Since this result is not explainable, equation 10 is preferred over equations 9 and 11.

In equation 10 the length from the overfall to

the watershed divide has been used to measure the effect of the watershed area above the overfall; this watershed area contributes runoff for the elongation of the gully. The gully length (X_8) has been included in equation 10, which is a measure of the watershed area contributing runoff to the perimeter of the gully. This length also gives an indication for potential gully growth through widening of the gully. With an increase in area terraced, the terrace variable reduces the gully surface area. This would be expected, since level terraces reduce the volume of runoff. The two remaining variables — X_{14} and X_3 — are both negative. Every regression that was made in the gully study shows X_3 to be negative. Engelstad (*3*) has given a possible explanation of the negative sign. He has observed large cracks in the areas of loess soil during extremely dry periods. Shrinkage cracks that form parallel to the gully sides would intercept surface runoff and tend to increase the rate of gully bank caving. Also, land cover is poorer, particularly in pastures, during periods of rainfall deficiency. The variable for the length from the gully overfall to the watershed divide, with its negative sign, is a subtractive factor that reduces the effect of X_8 when the gully is starting. As the gully length (L_1) increases, the value of X_{14} increases and approaches a maximum value equal to one.

In addition to statistical parameters and tests, it is possible to examine the predicted values and use them as a guide in selecting the most appropriate equation. The predicted values are obtained by substituting the original sample values of the independent variables into a given equation. In figure 1, the predicted value ($\hat{y}$) has been computed for equation 8. Since the original value of the dependent variable is represented by y, a value of 1 for the ratio y/y represents a perfect fit for a given sample value. In figure 1, a perfect fit for all sample values would be represented by a vertical line from the abcissa value of one. However, a perfect fit was not obtained and lines have been drawn on figure 1 to bracket the percentage of the sample

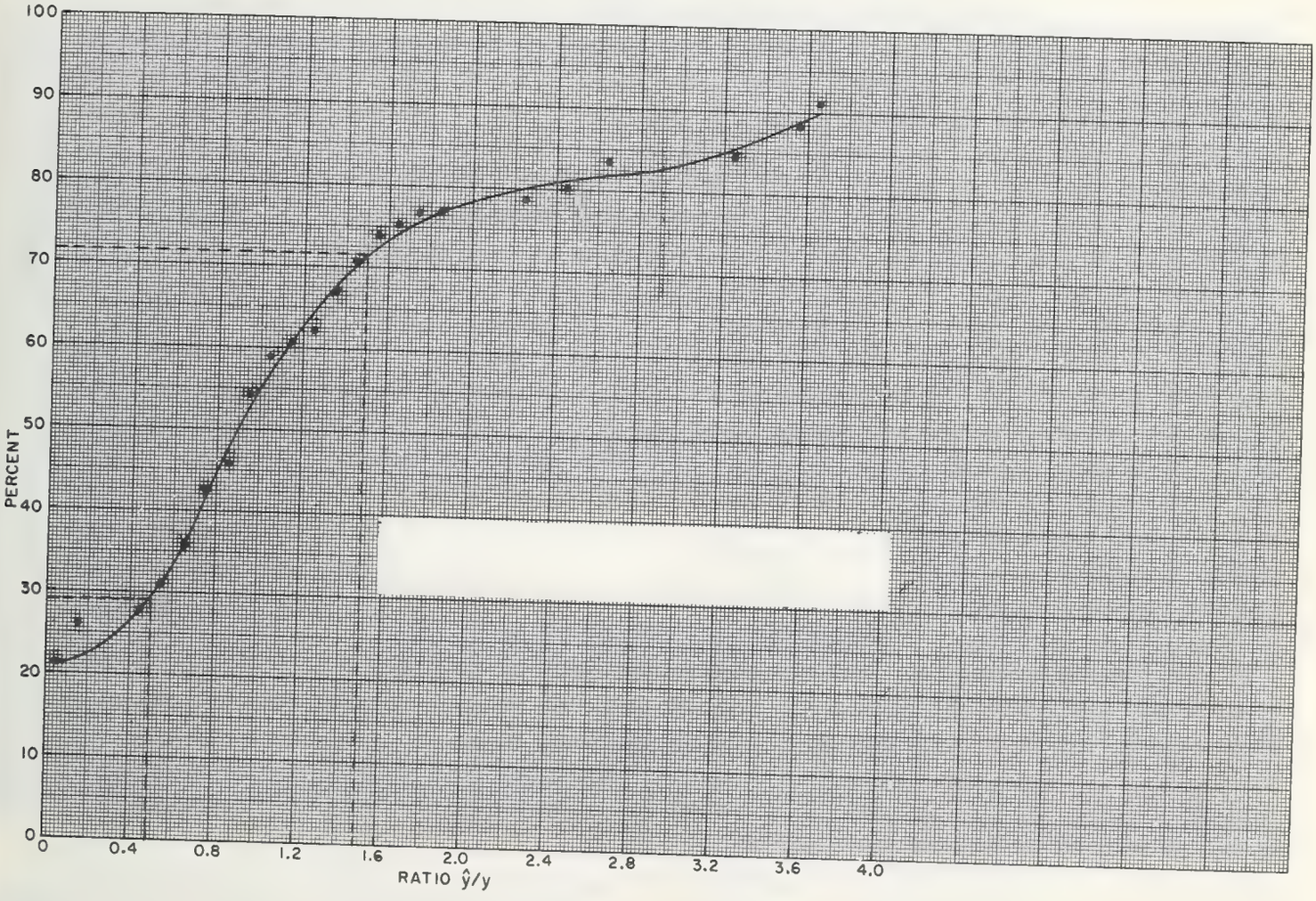

FIGURE 1. — Cumulative-distribution of the ratio of predicted value ($\hat{y}$) to original value (y) for linear equation No. 8.

values that were predicted within one-half of the original value. With the linear equation (equation 8) about 42 percent of the samples were predicted within 50 percent of the actual original value. The curve also shows that about 20 percent of the sample values were predicted with a negative value.

A similar comparison for the logarithmic equation (equation 10) is shown in figure 2. With the logarithmic equation, the percentage of samples predicted within 50 percent is increased to about 50 percent; an increase of 8 percent over the linear equation. It is also shown that no negative predicted values were obtained with the logarithmic equation.

Summary and Conclusions

The major objective of this study was to define a functional relationship that describes the gully development phenomena in western Iowa. Since no controlled studies of individual components responsible for the gullying process have been made, this study was based on a historical approach where all variables were evaluated from the past growth of gullies. Steer Creek Watershed, a gullied area in Harrison County, Iowa, was used for this study. The rates of gullying were determined with the use of controlled aerial flights, supplemented with a topographic survey that was made on the watershed 20 years ago. The hydrologic and watershed factors that were postulated to effect gullying were evaluated for the same period as for the gully growth.

The major emphasis in this study was directed to the evaluation of the lateral gully development since 1938. The gully, hydrologic, and watershed data were programmed for the IBM 650 computer. From the programmed data, prediction equations based on two models were obtained for the change in gully surface area. These models were the characteristic linear model and a logarithmic model where the independent factors are multiplied in the prediction equation. Although different hydrologic, gully, and watershed variables and combinations

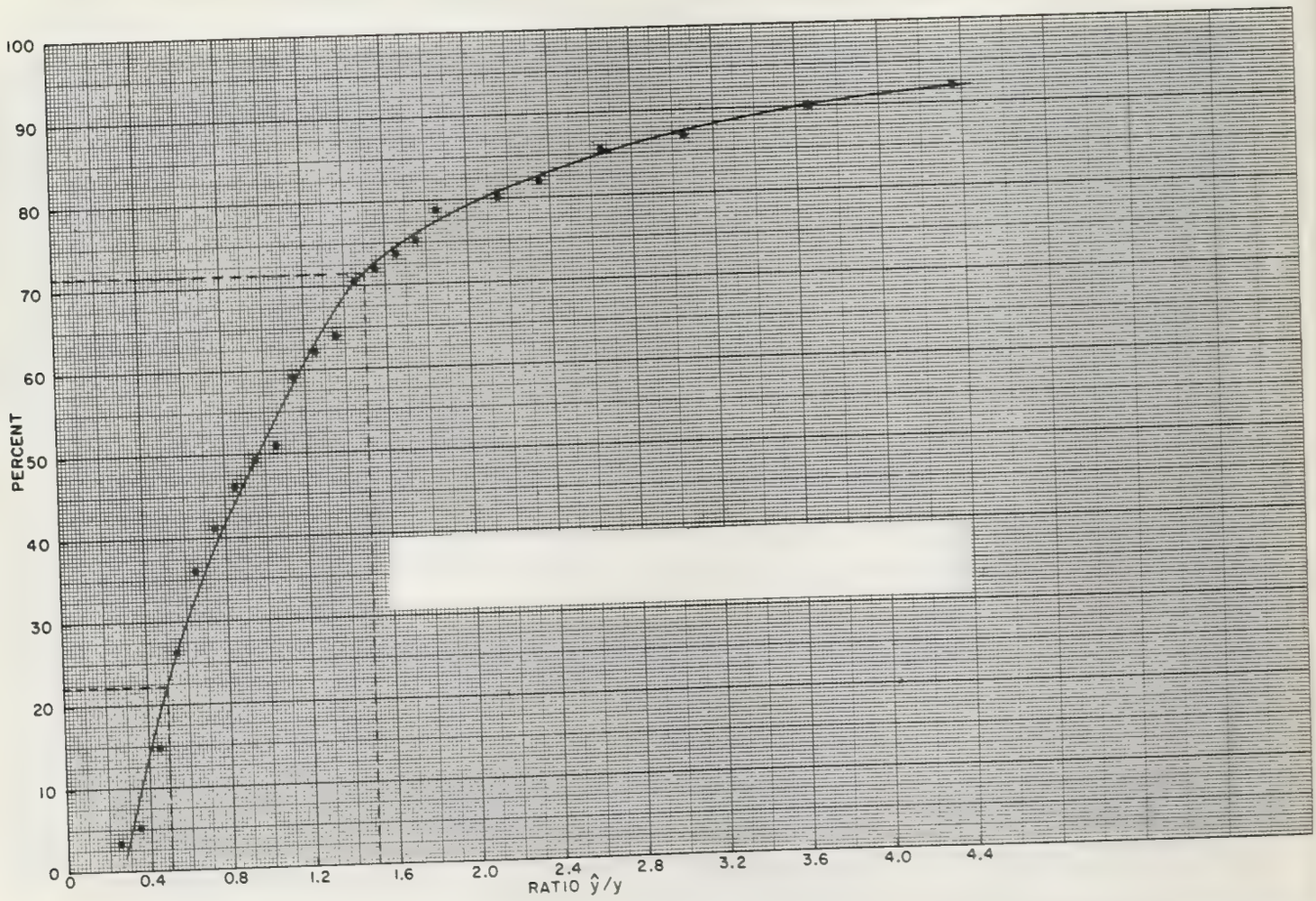

FIGURE 2. — Cumulative-distribution of the ratio of predicted value ($\hat{y}$) to original value (y) for logarithmic equation No. 10.

of these variables were used, the prediction equation for change in gully surface area that most nearly represented the gully development phenomena was:

$$X_1 = 0.01\ X_4^{0.0982}\ X_6^{-0.0440}\ X_8^{0.7954}\ X_{14}^{-0.2473}\ e^{-0.0360 X_3}$$

Where X_1 = Change in gully surface area, in acres;
X_3 = Deviation of precipitation from normal, in inches;
X_4 = Index of surface runoff, in inches;
X_6 = Terraced area of watershed, in acres;
X_8 = Gully length (L_1) at beginning of period, in feet;
X_{14} = Length from end of gully to watershed divide, in feet.

The conclusion that the functional relationship for the gullying process is a logarithmic relationship was supported by the fact that the average deviations from the fitted curve were smaller for the logarithmic model than for the linear model.

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LOGGING AND EROSION ON ROUGH TERRAIN IN THE EAST

[Paper No. 5]

By HOWARD W. LULL and K. G. REINHART, *Northeastern Forest Experiment Station, Forest Service*

During the past 150 years the forest lands in most of the southern Appalachian, Allegheny, White, and Green Mountains in the eastern United States have been cut over one, two, or more times. In the New England mountains, extensive operations began after the introduction of the steam-powered sawmill around 1850. Cutting reached a peak around 1880. The Allegheny and southern Appalachian forests were cut over somewhat later, and production reached its peak there about 1900. Cutting in virgin stands often followed a two-step procedure in which selected valuable species were cut first, followed by a second cutting a few years later in which everything salable was taken. Generally most of this forest land has been cut over every 30 to 60 years.

The results of the several cuttings are evident; forests now covering the Eastern mountains are second or third growth, and little remains of the original cover except isolated stems that, at time of logging, were either too hard to get to or were not worth cutting. A few scattered tracts of virgin timber are so rare that they have become tourist attractions, and well-worn paths attest to the popularity of these areas and portend their probable eventual demise due to soil compaction.

To the trained eye, evidences of the original logging are still visible in the occasional opening that bears the remnants of a logging camp, and, most of all, in the overgrown but still traceable logging roads that crisscross these areas and are now traveled only by the occasional hunter. These roads and the associated skid trails may well have occupied 10 to 20 percent of the logged-over area. Their aggregate length is great; in the upper Potomac river watershed, for instance, there is an estimated 1,620 miles of abandoned logging roads.

Logging Erosion

What has been the effect of this logging on erosion? General observation indicates that forest areas that have not been cut over in the past 20 or more years rarely show any signs of active erosion. The forest floor is covered with an inch or two of litter overlying 1 to 3 inches of humus (*6*). Even on steep slopes the litter shows no evidence of disturbance by overland flow, which strongly suggests that infiltration rates exceed rainfall intensities.

Most abandoned logging roads are stabilized. Generally they bear a mixed cover of seedlings, saplings, shrubs, herbs, moss, and litter; and some have an erosion pavement.

Streamflow from the forested headwaters is generally clear and prized for its purity; the second- and third-growth forest apparently is an excellent guardian of our water supplies. Under heavy rainfalls, the water becomes discolored — but more by organic debris or from

bank cutting than by sediment from the tree-covered slopes.

For example, turbidities of streamflow from the second-growth forested watersheds of the Coweeta Hydrologic Laboratory in western North Carolina are generally less than 2 p.p.m. (parts per million) during nonstorm periods and they range well under 11 p.p.m. (the drinking water standard) during most storm periods. Extreme conditions may produce turbidities as high as 80 p.p.m., most of it organic matter picked up by the high flow (*1*). Streamflow from second-growth watersheds at the Fernow Experimental Forest in West Virginia has an average turbidity under 5 p.p.m. (*8*). At the Hubbard Brook Experimental Forest watersheds in New Hampshire, turbidities during nonstorm periods are less than 1 p.p.m.; during an unusually heavy storm they have ranged from 4 to 11 p.p.m. These forested watersheds have not been disturbed by logging for several decades.

Such conditions are somewhat difficult to square with an opinion held by many that forest cutting leads to erosion, sediment in the streams, and associated land ills. Forest watershed research results back up this opinion with some evidence. For instance, at the Coweeta Hydrologic Laboratory, exploitative logging resulted in a maximum turbidity of 5,700 p.p.m. (*1*); and at the Fernow Experimental Forest, a maximum turbidity of 56,000 p.p.m. (*8*). Keys to this paradox may lie in certain relationships recently observed on the Fernow Experimental Forest where four small forested watersheds were logged according to four different practices.

The Fernow Studies

The Fernow experimental watersheds, ranging in size from 38 to 96 acres, had before treatment a forest cover of second-growth hardwoods about 50 years old interspersed in places with a large number of old residuals left from original logging. Principal species were red and chestnut oaks, black cherry, yellow-poplar, sugar maple, beech, birch, and basswood. The soils of these watersheds are strongly acid (pH 4.5 to 5.0) silt loams, primarily of the Calvin series. Soil depth to bedrock ranges from 25 to 48 inches with an average depth of 30 inches. Stone content is moderately high. Bedrock consists of interbedded red shales and sandstones of the Catskill series. Slopes are steep, averaging 40 to 65 percent, and logging is difficult.

After a calibration period, four of these watersheds were cut over in 1957-58, and a fifth was left uncut as a control. Turbidity samples were collected before, during, and after logging; and ring-infiltrometer tests were made on and off the skidroads.

Watershed treatments ranged from a commercial clear-cutting with logger's choice skidroads to a conservative selection cutting with carefully planned and constructed skidroads (table 1). Logging was done with a TD–9 crawler tractor equipped with a rubber-tired sulky and winch. Generally the tractor stayed on the skidroads and winched up the material in tree lengths. Tractor skidroads — but no truck

TABLE 1.—*Forest practices applied to Fernow experimental watersheds*

Practice	Timber cut	Period of logging	Maximum [1] of grade skidroads	Water bars	Other requirements
Commercial clear-cut.	Everything merchantable: 8.5 M bd.-ft. per acre.	May 1957 to June 1958.	No restrictions...	None.....	None.
Diameter limit.....	All marketable trees over 17 inches d.b.h.[2]: 3.9 M bd.-ft. per acre.	June to Aug. 1958.	No restrictions...	At 2-chain... intervals.	None.
Extensive selection...	Selected trees above 11 inches d.b.h.: 2.3 M bd.-ft. per acre.	Aug. to Nov. 1958.	20 percent.....	As needed...	No skidding in streams.
Intensive selection....	Selected trees above 5 inches d.b.h.: 0.9 M bd.-ft. per acre.	Oct. 1958 to Feb. 1959.	10 percent.....	As needed...	No skidding in stream. Skidroads located away from streams. Grass seeding for soil stabilization where needed.

[1] To be exceeded only for short distances when necessary.
[2] D.b.h. = diameter at breast height (4.5 feet above ground).

roads — were constructed on the watersheds. Area and grade of bulldozed skidroads are given in table 2. Greater care in planning reduced both the area of skidroads and their grades.

TABLE 2.—*Proportion of skidroad, by area and grade*

Practice	Percentage of watershed in—		Percentage of bulldozed skidroad, by grade class—			
	Bulldozed skidroads	Bulldozed and nonbulldozed skidroads	0–10 percent	11–20 percent	21–30 percent	31–40 percent
Commercial clear-cut	3.6	7.3	22	32	35	11
Diameter limit	2.5	6.2	20	72	8	0
Extensive selection	2.1	5.8	36	57	7	0
Intensive selection	.8	1.9	68	31	1	0

Maximum turbidities occurred during the logging operation, and they ranged from 56,000 to 25 p.p.m. (table 3). The range of values and

TABLE 3.—*Maximum turbidities measured and frequency distribution of turbidity samples during the logging operation*

Cutting practice	Maximum turbidity	Samples in turbidity unit classes				
		0–10 p.p.m.	11–99 p.p.m.	100–999 p.p.m.	1000+ p.p.m.	Total
	P.p.m.	*Number*	*Number*	*Number*	*Number*	*Number*
Commercial clear-cut	56,000	11	7	11	11	40
Diameter limit	5,200	5	5	8	7	25
Extensive selection	210	14	0	1	0	15
Intensive selection	25	22	1	0	0	23

the frequency distribution of samples are clearly related to logging practices. Before logging, streamflow from all the watersheds had average turbidities less than 5 p.p.m.

Here we should note that some logging operations, using different equipment and methods, result in much greater skidroad area and greater disturbance of the forest floor than occurred even on the clear-cut watershed on the Fernow Forest.

Sources of sediment at the Fernow Forest were improperly constructed skidroads and skidroads located too close to the streams. Ring infiltrometer tests made both on and off these roads (table 4) indicated limited infiltration only in the tread area of bulldozed skidroads. However, the large amount of soil moved from the skidroads suggested that overland flow was greater than a comparison of rainfall and infiltration rates would lead one to expect. Subsurface flow, intercepted at the road cut, may

TABLE 4.—*Infiltration rates, in inches per hour, on and off skidroads*

Area and location No.	Infiltration rate in—		
	Tread area	Center, between treads	Adjacent undisturbed area
Bulldozed skidroad:			
1	6.4	5.1	96.8
2	.5	11.3	260.9
3	2.4	39.7	89.6
Mean	3.1	18.7	149.1
Non–bulldozed skidroad:			
4	37.3	15.2	49.6
5	5.7	11.5	109.1
6	9.8	18.5	66.7
Mean	17.6	15.1	75.1

have been a major contributor. Also, logging debris in the stream diverted stormflow from the channel to the road and this resulted in excessive erosion.

Erosion from Roads

Apart from logging and water-quality studies, a few other studies indicate the amount of erosion-on-site connected with logging operations. For instance, in the Ouachita Mountains of Arkansas, a survey of skid trails, logging roads, and concentration yards showed that a commercial clear-cutting operation laid bare at least 2.5 percent of the area and caused 3.6 tons of topsoil to be eroded per acre; on a selectively cut area, 1.2 percent of the watershed was affected, and less than 1.5 tons of soil per acre was lost (*3*).

A study at Coweeta showed an average soil loss of 1.3 inches in a 3-month period from a road with an average grade of 30 percent (*4*). This loss is of the same general magnitude as first-year erosion losses reported from skidroad studies at the Fernow Forest (*11*).

Thus, extensive erosion *can* result from logging, and it has so resulted in the past. However, erosion from a logging road and damage to water quality are not one and the same. For, when the road is located some distance from a stream, turbid road runoff usually does not reach the channel; the water sinks into the forest floor, leaving the sediment on the surface. But, located close to the stream, the road can feed sediment directly into the channel. Road location obviously is the most important factor in water-quality control during logging.

After-Logging Erosion

After logging on the Fernow Forest, water-quality measurements indicated a rapid reduction in erosion (table 5). An important cause

TABLE 5.—*Percentage of samples by turbidity classes*

Cutting practice	Period	Turbidity classes				Samples
		0–10 p.p.m.	11–99 p.p.m.	100–999 p.p.m.	1,000+ p.p.m.	
		Percent	*Percent*	*Percent*	*Percent*	*Number*
Commercial clear-cut	During logging	27.5	17.5	27.5	27.5	40
	After logging:					
	1st year	53.5	31.4	12.8	2.3	86
	2d year	90.9	6.8	2.3	.....	87
	3d year	95.1	4.9	.....	.....	81
	4th year	94.4	5.6	.....	.....	72
Diameter limit	During logging	20.0	20.0	32.0	28.0	25
	After logging:					
	1st year	87.8	12.2	.....	.....	81
	2d year	100.0	.....	.....	.....	79
	3d year	100.0	.....	.....	.....	79
	4th year	100.0	.....	.....	.....	57
Extensive selection	During logging	93.3	.....	6.7	.....	15
	After logging:					
	1st year	96.3	3.7	.....	.....	81
	2d year	100.0	.....	.....	.....	84
	3d year	98.8	1.2	.....	.....	80
	4th year	100.0	.....	.....	.....	37
Intensive selection	During logging	95.7	4.3	.....	.....	23
	After logging:					
	1st year	100.0	.....	.....	.....	85
	2d year	98.8	1.2	.....	.....	84
	3d year	100.0	.....	.....	.....	81
	4th year	100.0	.....	.....	.....	15

was the cessation of skidding that heretofore had loosened the surface soil of the skidroad and had compacted the soil below the surface. The development of an erosion pavement was another factor: the soils contained about 50 percent stone fragments by volume and quickly developed a protective stone cover. The rapid development of vegetation, woody and herbaceous, coupled with leaf fall, served further to reduce erosion.

Of these several factors, the formation of erosion pavement was the most important. Grant and Struchtemeyer (*2*) have attributed three beneficial effects to erosion pavement in promoting infiltration and reducing runoff and erosion: erosion pavement acts as a rock mulch by intercepting and dispersing rain-drop energy; it reduces the detaching action of flowing water; and it is associated with an increased noncapillary porosity.

Where soils are not stony, erosion can continue for some time, as at Coweeta where for several years clay subsoil from skid trails and roads continued to move into streams after every storm. Three years after the eroding areas were stabilized by seeding, the water was still slightly murky during normal flows.[1] As a general condition, however, soils of the Allegheny Mountain region, and particularly in the glaciated regions, are stony or have a high content of coarse fragments.

This combination of factors — erosion pavement, rapid invasion by vegetation, leaf fall — and the facts that only a small proportion of any logging operation has exposed soils and that the naturally high infiltration rates of forest soils are not affected by logging are probably the reasons why the forested watersheds of our eastern mountains, though heavily cut over in the past, can generally still provide adequate water-quality protection.

Watershed-Management Applications

This self-healing process however, may be effective only after considerable damage to water quality has been done; and in some areas, such as the Coweeta example, the reduction of water quality may continue for some time. This need no longer be; for from studies at the Fernow Forest and at other areas, a number of principles and practices for water-quality protection have been developed.

For instance, excessive damage to water quality from roads running close to the stream can be avoided by careful road location. As a general rule, the minimum distance of logging road from a stream should be 25 feet plus 2 feet for each percent of slope between road and stream (*10*).

Also important is the grade and drainage of the road. On a steep road without provision for drainage, accumulated surface runoff carrying

[1] Personal communication from John D. Hewlett, project leader, Coweeta Hydrologic Laboratory, Jan. 8, 1963.

a high proportion of soil may discharge directly into the stream at the bottom of the slope. Even if discharge is not directly into the stream, the volume and sediment content may be such that the water cannot infiltrate into the forest floor over any reasonable distance.

If the roads in the two Fernow watersheds that produced the highest sediment yields had been properly located and drained, maximum turbidities in parts per million would have been in the low hundreds rather than in the 5 to 50 thousand range. Care in location and construction should not burden the logger, for it pays off in efficiency and reduced logging costs (*5, 7, 9*).

Most of the erosion from logging roads occurred during the logging operation. This suggests:

1. That the operation in any one area should not be prolonged, but should be completed as soon as possible.

2. That more attention should be paid to preventing erosion during the operation. It is not enough to limit erosion control measures to afterlogging care. Perhaps the most practical measure is to cut and maintain broad-based outsloped drainage dips across skidroads. This is not always easy, and the idea will often be resisted by loggers.

This study points up again the fact that erosion from only a fraction of the logging area can pollute a lot of water. Hoover (*4*) has pointed out that a short stretch of logging road can produce much more sediment than occasional patches of steep land in cultivated crops. The forester who might not permit clearing a piece of forested municipal watershed for a row crop because of the erosion hazard should feel just as much concern over the location of logging roads.

Finally, observations in many areas indicate that continuously used permanent road systems in the forest can create serious water-quality problems. Standards for constructing and maintaining such roads should be even higher than for logging roads that are used for only short periods of time.

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SEDIMENT YIELDS FROM SMALL WATERSHEDS UNDER VARIOUS LAND USES AND FOREST COVERS[1]

[Paper No. 6]

By S. J. Ursic, *research forester, Southern Forest Experiment Station, Forest Service,* and Farris E. Dendy, *agricultural engineer, USDA Sedimentation Laboratory, Soil and Water Conservation Research Division, Agricultural Research Service*

Reported here are 3-year results of studies that two agencies of the U.S. Department of Agriculture are conducting on small upland watersheds in northern Mississippi. The Agricultural Research Service has installations on crop and pasture lands. The U.S. Forest Service is studying lands that are now in forest or which, because of progressive erosion, have been retired from agriculture and are suited only for forests that will protect watersheds and produce timber.

[1] Research cooperative with the University of Mississippi and Mississippi State University.

Description and Methods

The hill lands of the upper Coastal Plain in northern Mississippi were originally forested

with pine and hardwoods. When cleared for agriculture, most of them eroded severely. Erosion is now so widespread that hydrologic studies of specific soil–vegetation combinations must be confined to small headwater catchments. The 16 watersheds in this study were selected to represent prevailing conditions of soils, slopes, erosion, land use, and plant cover.

Land Uses and Cover Types

The watersheds include one in pasture and three replications each of lands in cultivation, abandoned fields reverting to native grass, depleted stands of upland hardwoods, abandoned lands planted to loblolly pine, and forests of mature shortleaf pine and hardwoods. The drainages range from 1.5 to 4.5 acres (table 1).

TABLE 1.—*Physical characteristics of study watersheds*

Land use or cover type	Drainage area	Soils		Range in elevation
		Loessial[1]	Coastal Plain[2]	
	Acres	*Percent*	*Percent*	*Feet*
Corn	3.88	100	0	27
	1.61	100	0	18
	1.45	58	42	21
Pasture	3.01	62	38	28
Abandoned fields	2.65	100	0	37
	2.62	64	36	44
	2.43	25	75	49
Depleted hardwoods	2.56	65	35	49
	2.12	34	66	58
	2.13	100	0	44
Pine plantations	3.35	29	71	68
	3.58	46	54	60
	2.60	100	0	40
Pine–hardwoods	3.31	16	84	74
	4.56	6	94	99
	4.01	4	96	95

[1] Providence, Lexington, Loring, and Grenada series.

[2] Principally Ruston soils (Wilcox on pine–hardwood watersheds).

The pasture has been heavily grazed since 1957.

The three cultivated watersheds are planted to corn each year. One receives preferred management practices consisting of high rates of fertilization, high plant populations, and cultivation on the contour.

The abandoned fields support a grass-herbaceous cover dominated by broomsedge (*Andropogon* spp.). Once planted continuously to cotton, they were abandoned as a result of progressive erosion and waning production. They have not been burned or grazed in recent years; cover is near maximum for the type and site.

The upland hardwood forests, depleted by a century of overcutting, grazing, and frequent wildfire, now are sparse stands of poor quality and low commercial value. Blackjack oak (*Quercus marilandica* Muenchh.), post oak (*Q. stellata* Wangenh.), and hickory (*Carya* spp.) are the principal species. The forest floor lacks the high degree of biotic activity that creates desirable hydrologic properties in some hardwood soils.

The loblolly pine plantations were established in 1939 on fields from which erosion had removed an estimated 2 feet of the surface and had cut gullies 5 feet below the level of the remaining soil. Loblolly pine is the main species for erosion-control planting in northern Mississippi. It has been established on 350,000 acres; planting the pine on abandoned land and to convert stands of depleted hardwoods to pine continues at the rate of 40,000 acres a year.

The pine–hardwood forests are on land too steep for cultivation, but for many years they were subjected to grazing, heavy cutting, and frequent wildfire. Since 1936 they have been protected from these influences as part of the Holly Springs National Forest; their stocking now averages about 5,000 board feet of shortleaf pine and 600 board feet of hardwoods per acre.

Soils

Two major groups of Red-Yellow Podzolic soil are represented: Loring, Providence, and Lexington series derived from wind-deposited loess; and Ruston and closely allied series developed from Coastal Plain materials. The loess soils are primarily silt loams; the Coastal Plain soils, sandy loams. On two of the cultivated fields and on one each of the abandoned fields, depleted hardwood stands, and pine plantations, all soils are loess. The remaining watersheds representing these covers, together with the pasture watershed, have Coastal Plain soils on the lower slopes and loess on the upper slopes and ridges.

Ruston soils are important agricultural and forest soils in all the Atlantic and Gulf Coast States. Loessial soils are prevalent in all States bordering the Mississippi River from Illinois southward.

Soils on the pine–hardwood watersheds are primarily Wilcox sandy loams shallowly underlain by a layer of firm mottled clay that restricts internal drainage. Because of this layer, the soils are hydrologically shallow and are not comparable to those on the other watersheds. Total runoff is greater than from the other forest covers, but it is released gradually so there is measurable flow for up to 6 months each year.

Instrumentation

Runoff from the pasture and cultivated watersheds is measured with modified Parshall flumes. Sediment is collected in concrete boxes equipped with slot-type samplers for sampling

overflows.[2] Quantities of sediment are determined monthly.

Runoff from the abandoned fields and forested watersheds is measured with 3-foot H-type flumes. Sediment yields are determined for individual storms. Deposited sediment is collected in a concrete approach section. Samples for suspended-sediment determinations are obtained with a Coshocton wheel sampler.[3] All flumes are equipped with FW–1 water-stage recorders.

Precipitation

Average annual precipitation in the study region is 52 inches. Average rainfall on the watersheds was 50, 44, and 61 inches in 1959, 1960, and 1961, respectively. Sixty-one inches is exceeded only about once in 5 years. The 3-year period provided opportunity to observe the interplay between cover, soils, runoff, and sediment production under a representative range of precipitation.

Results and Discussion

Sediment production decreased in the order: corn > pasture > abandoned fields and depleted hardwoods > pine plantations and mature pine-hardwoods (table 2). The maximum annual yield of sediment from a single watershed among the 12 representing forest land covers was 0.54 ton per acre. The minimum from the pasture exceeded this amount by a factor of 2, that from the cultivated fields by a factor of 6. Although comparisons between the abandoned field-depleted hardwood types and the two pine covers were less clear-cut, differences in mean annual sediment yields were significant in 1960 and highly significant in 1961. In 1959 differences in yield between the two cover groups were not significant; the lack of significance was due largely to sediment from a single storm in which 5 inches of rain fell on the pine plantations but less than 3 inches on the other covers.

TABLE 2.—*Sediment and surface water yields*[1]

Land use or cover type	Average annual rainfall	Average annual runoff	Annual sediment yields	
			Means	Ranges
	Inches	*Inches*	*Tons per acre*	*Tons per acre*
Open land:				
Cultivated	52	16	21.75	3.28– 43.06
Pasture (one unit)	51	15	1.61	1.19– 2.03
Forest land:				
Abandoned fields	51	7	.13	.01– .54
Depleted hardwoods	51	5	.10	.02– .32
Pine plantations	54	1	.02	.00– .08
Mature pine–hardwoods[2]	51	9	.02	.01– .04
Gullies[3]	53	—	182.	84.3 –399.3

[1] Data are means of 9 values, 3 replications of each cover for the 3 years, 1959–61 except pine–hardwoods (1960–61).
[2] These watersheds are on hydrologically shallow soils.
[3] Average annual rainfall and sediment outflow from 7 gullies for the 5 years, 1956–60. (See text footnote 4.)

Annual sediment yields from pine plantations and pine-hardwoods averaged less than 50 pounds per acre. This amount is probably not in excess of the geologic norm for undisturbed native forests of the area.

Loessial soils, although highly erosive, did not consistently yield the most sediment. Of the pine plantations, the one on loess soil had the highest sediment rate, but the all-loess abandoned field had the lowest rate in its group. The highest yield from the depleted hardwoods was from the watershed that had the highest proportion of sandy soils. On the cultivated watersheds, soil losses were highest from the two all-loess units, but soil effects were confounded with the improved practices on the unit having both types of soil.

Added to table 2 for comparative purposes are data from seven individual gullies studied by Miller and others.[4] The gullies, which have drainage areas of 0.32 to 0.64 acre, are within a few miles of the small-watershed installations and are formed on similar soils. Annual sediment yield from the gully with minimum erosion was twice the maximum from the cultivated fields and 156 times greater than the maximum from the forest watersheds. The gully with maximum erosion was annually losing soil at the rate of 400 tons per acre, or 2.36 area inches.

Contributing to differences in sediment yields among covers were varying amounts of annual runoff and sediment concentrations.

Precipitation-Runoff Relationships

The effect of land use and cover types on annual runoff is shown in figure 1. The data indicate discrete populations with runoff decreasing in the order: corn and pasture > abandoned fields and depleted hardwoods > pine plantations. As the pine-hardwood watersheds are on entirely different soils, they are not represented in figure 1. If plotted, their values would fall within the range shown for abandoned fields and depleted hardwoods.

For each cover type, runoff was greatest from the watershed with all loess soils. Despite variation due to soils, however, runoff from the cultivated and pasture watersheds and from the abandoned field and depleted hardwood covers

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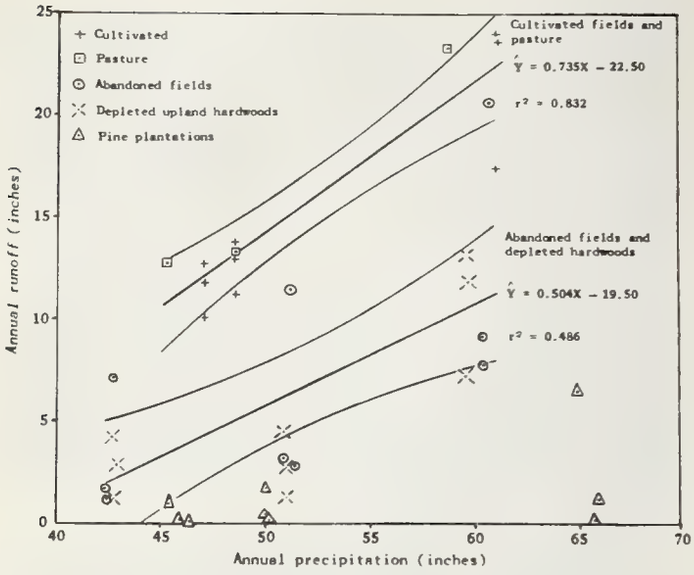

FIGURE 1. — Effect of land use and cover types on the precipitation-runoff relationship for small watersheds in northern Mississippi. Plotted points are annual values for individual watersheds for 3 years, 1959-61. Regressions for the cultivated fields and the pasture, and for the abandoned field and depleted hardwood covers, are shown with 95-percent-confidence limits.

was significantly correlated with annual precipitation.

While runoff from the pine plantations was not significantly correlated with annual precipitation, this cover represents a third population of runoff potential. Mean annual runoff from the pine plantations was significantly less (5-percent level) than from the abandoned fields and the depleted hardwoods, despite higher rainfall on the plantations than on the other covers.

Sediment Concentrations Varied by Cover Types

Sediment concentrations also decreased in the order: corn > pasture > abandoned fields and depleted hardwoods > pine plantations and pine-hardwoods. The minimum average concentration per acre-inch of annual runoff from a cultivated watershed was 0.32 ton. The maximum from the pasture was 0.12 ton.

The lowest annual average concentration from the pasture (0.087 ton) exceeded the maximum of the 18 values for abandoned fields and depleted hardwoods (0.084 ton).

Since runoff amounts from the cultivated fields and pasture were similar (table 2), differences in sediment yields from the two covers were due largely to sediment concentrations. The grass cover reduced soil movement. Runoff from the pasture, however, was higher than from the watersheds in native grass; and the excess contributed to higher sediment yields. This was perhaps the result of the accumulation of less litter on the pasture, and the compaction of soils by animals, which reduced infiltration and caused higher rates of overland flow.

Average annual sediment concentrations per acre-inch of runoff from the abandoned field-depleted hardwood watersheds (0.022 ton) were twice as great as from the pine plantation and pine-hardwood covers (0.011 ton). The difference fell short of statistical significance. Since three-fourths or more of the annual sediment yields came from a few key storms, seldom more than 5, concentrations from these storms were compared. It was first established that sediment concentrations were not significantly correlated with amounts of overland flow on either of the two cover groups. Overland flow rather than total runoff was used in this comparison because of the very low concentrations of sediment during protracted periods of low flow. Such flows occurred on watersheds with a high proportion of loessial soils and on watersheds covered with pine-hardwoods. Overland flow for individual storms was estimated as that part of the hydrograph above a straight line connecting the beginning of the rise to the point of maximum curvature on the recession. The mean concentration of sediment per acre-inch of overland flow from the abandoned fields and depleted hardwoods for these storms was 122 pounds, as compared to a mean concentration of 46 pounds from the pine and pine-hardwood areas. The difference between these means was significant at the 1-percent level.

Sediment-Precipitation Relationships

General trends, although they did not achieve significance, indicated that annual sediment yields from various covers increase directly with annual precipitation. Future studies of individual storms should help remove the masking effect of varying runoff from individual watersheds, differing storm characteristics, and variations in sediment concentrations. Cover, however, because it directly affected both runoff and sediment concentrations, was the dominant influence in determining sediment yields (fig. 2).

Figures 1 and 2 indicate that considerable reduction in runoff and sediment can be achieved by changes in land use or cover types. In the future it is planned to change the cover on several of these watersheds to confirm and further refine these findings.

Summary

Data from small watersheds in the hilly uplands of northern Mississippi show large variations in annual runoff and sediment production attributable to land use and cover types. Runoff decreased in the order: corn and pasture > abandoned fields and depleted hardwoods > pine plantations. Annual sediment yields and average concentrations of sediment per unit of run-

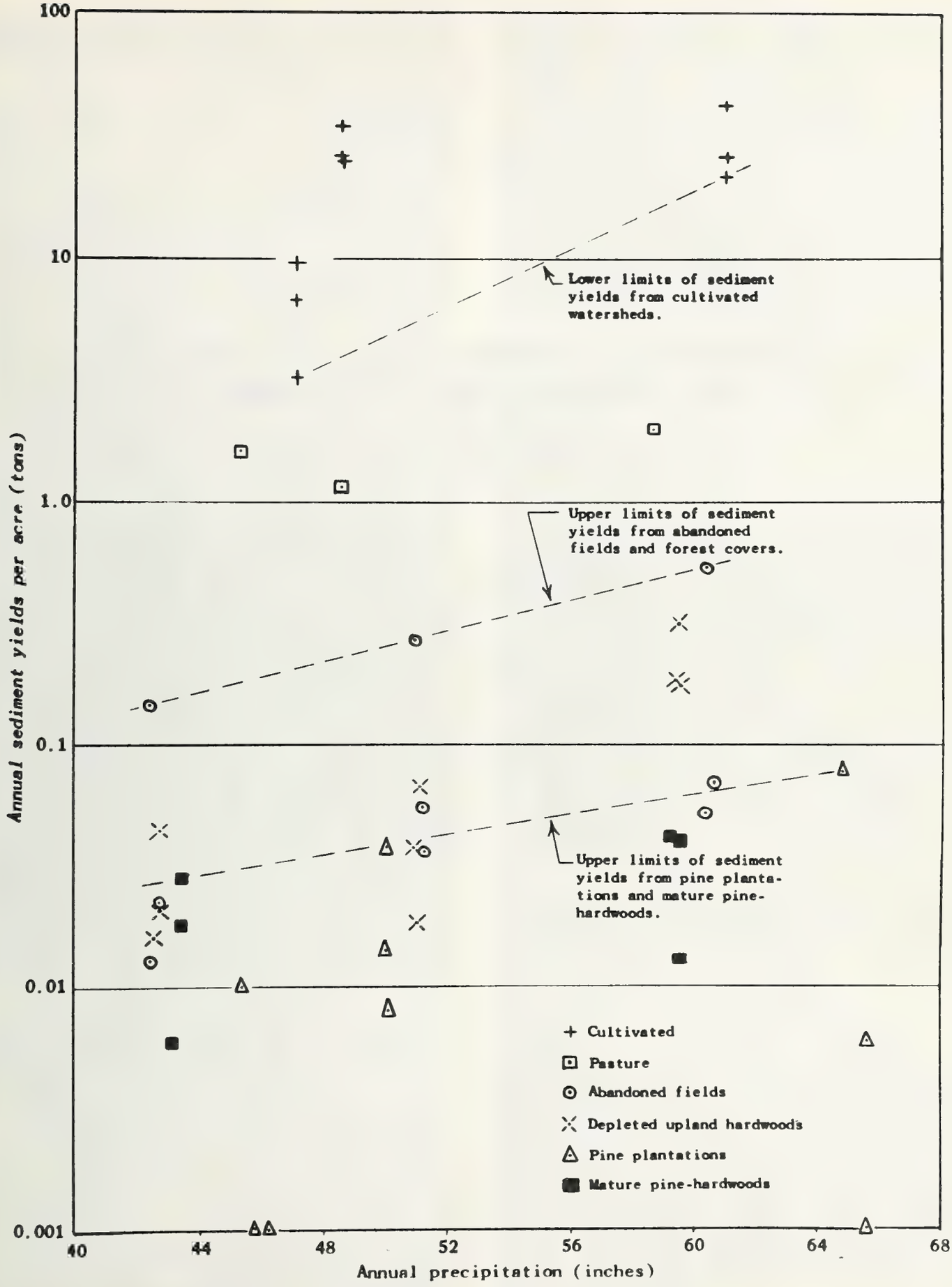

FIGURE 2. — Annual precipitation and sediment yield from individual watersheds in northern Mississippi, 1959-61.

off decreased in the order: corn > pasture > abandoned fields and depleted hardwoods > pine plantations and mature pine-hardwoods. These progressions represent discrete populations of erosion potential.

Runoff was greater from watersheds with loessial soils than from those with both loess and Coastal Plain soils, but the effect of soil on sediment yields was not consistent for all covers.

Extremes in annual sediment production ranged from 43 tons per acre from a cultivated watershed to a few pounds per acre from pine plantations. Sediment yields from abandoned fields with a dense cover of native grass and from forest covers did not exceed 0.5 ton per acre annually. By contrast, yields from gullies in the same locality have been reported as 84 to 400 tons per acre.

The studies are yielding data that should eventually allow prediction of sediment production from permanent covers. They suggest opportunities for reducing runoff and sediment by changing land use and cover types.

Establishing pine on actively eroding abandoned fields has in two decades reduced sedimentation to amounts probably not in excess of the geologic norm for undisturbed climax forests in this area.

EFFECT OF HIGHWAY CONSTRUCTION AND MAINTENANCE ON STREAM SEDIMENT LOADS

[Paper No. 7]

By W. E. Bullard, *forester, Watershed Management, Division of Water Supply and Pollution Control, U. S. Public Health Service*

Thousands of miles of freeway, highway, country road, and forest access roads are built in the United States every year. Nearly every foot of this construction is in a watershed, and, except for overpasses and viaducts and bridges, nearly every foot of it involves soil disturbance. Whenever there is soil disturbance, there is a potential sediment source. Too often in highway construction this potential is realized; disturbed soil erodes and erosion products are carried to streams and become damaging sediments. Maintenance operations subsequent to construction often accelerate the process.

This paper describes the adverse effects of road construction and maintenance on stream sediment loads and points out how to avoid or reduce them.

Soil Disturbance

In more or less chronological order, the steps in building a road are clearing the right-of-way, bulldozing an accessory "shoo-fly" road, relocating stream channels, opening up the cuts, putting in culverts, hauling and dumping rock and soil in the fills, side casting or end hauling excess material not needed for fill, digging drainage ditches, smoothing and compacting the surface, and spreading and smoothing the final surfacing material. Each of these steps may, and often does, involve unnecessary soil disturbance and erosion. There are other changes: deep cuts may intercept ground water and dry out the slopes; imposition of a new drainage pattern may radically change local streamflow regime; and new openings in forest cover may modify the local ecology and microclimate.

Sidehill cuts and fill slopes are major sites for soil disturbance and erosion that provides stream sediments. On long, steep slopes directly above stream channels, the movement is immediate and rapid. End hauling is expensive and interferes with progress of the road; gravity is cheap and always ready to work. Solution of the sidehill-cut problem as a sediment source largely depends on avoidance. Sidehill locations, wherever possible, should be bypassed in favor of bench and ridge top locations that do not involve as much cut and fill. Or cribbing can be used to build up one side of the roadbed to reduce the volume of cut needed. Leaving a screen of brush and trees between the road and the channel below will trap much of the overcast debris that would otherwise become sediment.

Channel relocations that may appear necessary from the standpoint of road construction economics often initiate long-chain reactions that are felt for many miles downstream. Moving the stream to a rock cut may be perfectly safe; but changing its length and gradient on other than a solid bottom will inevitably start a cycle of bank cutting, meandering, and reworking of old deposits that adversely affect both water quality and aquatic habitat. Such upsets are particularly harmful in the upper reaches of streams, since these are usually the zones of supply and replenishment of food and fish populations for downstream reaches.

Where the road cut has removed the toe support to a slope, slides will occur until a new equilibrium is established on the slope. This can be avoided or reduced by draining the soil mass subject to movement or by providing cribbing or toe-wall support. Otherwise, maintenance is apt to be excessive for a considerable

period; and since maintenance generally consists of shoving the unwanted excess over the side of the road, erosion and consequent sedimentation are apt to be excessive also.

A well-rocked road surface will prevent considerable sedimentation. Without a good rock surface, roads tend to become rutted with use; ruts erode, intercept drainage, and carry fine sediments to diversion points. Even the splash from rutted roads along streams has a noticeable effect on stream turbidity. The best surface is broken rock that packs well to support heavy loads and yet remains porous enough to drain readily. In regrading a road surface, the object should be to redistribute the surface material evenly, not to scrape it off and shove it over the edge.

The places where the needed fill material and surfacing material come from may also be involved in sediment production. When more fill is needed than the cuts provide, it is usually taken from borrow pits; these borrow pits may become sores on the landscape that erode and put muddy drainage into streams. Borrow pits should be planned and operated so that the areas of bare disturbed soil do not erode into stream channels. Surface soil should be stockpiled, and when the job is finished, spread over the abandoned borrow pit, graded, mulched, and seeded or planted to naturalize and stabilize it against erosion. The same process also serves to reduce accident hazards and clean up mosquito breeding places.

Sharp, crushed hard quarry rock makes the best surfacing material, but it is not always available. River gravel is often closer at hand and cheaper. But taking out river gravel disturbs the channel, often sufficiently to initiate a cycle of meandering and bank cutting, and directly muddies the water where bulldozer and dragline work in the river itself. In streams of the Northwest, these operations cause serious damage to fisheries if done at a season when eggs and young fry are in the stream bottom gravels. Where gravel dredging in a live stream is necessary, it should avoid the spawning season. It may also be necessary to avoid the extremely low water season, as sediments stirred up may adversely affect temperature and oxygen content of the stream.

Disruption of Drainage Patterns

Since roads tend to be horizontal rather than vertical, they cross the natural drainage channels on the land. The road cuts often intercept ground water and bring it to the surface and road fills often block intermittent drainage channels and obstruct — to a greater or lesser degree — the main drainage channels. Resurrected ground water and the flashy runoff from road surfaces can overload natural channels and start a cycle of bank cutting that loads the stream with sediment and affects the acquatic habitat for long distances below.

Channel obstructions may arise from encroachment of fill slopes, bridge piers, poor placement of culverts, or culverts of inadequate capacity. They may also arise from careless right-of-way clearing where debris that rolls or falls into channels is left untouched or from the blasting of rock points and ledges that dump tons of rock into channels. Obstructions change the streamflow regime and often cause the stream to seek a new channel, a process that involves bank cutting and channel bed reworking with consequent sedimentation and loss of aquatic habitat.

New drainageways provided by uncontrolled road drainage turnouts that gully their way to the nearest natural channel are always a dependable source of sediment. And as the gullies work headward, they can readily undermine the road and increase maintenance needs. Maintenance by dumping material into the gully only continues the sediment production and does not cure the erosion "sore" nor solve the maintenance problem. Any gully also tends to drain off ground water and dry out the slope; while this may sometimes be desirable to reduce slumping and sliding, it generally is adverse to production of tree, forage, or other crops on the land affected.

Overloading of natural channels by the rapid concentration of road drainage water may have serious consequences in sedimentation. Road surfaces may account for as much as 3 percent of the area of a small upstream watershed; this impermeable 3 percent sheds water as fast as it falls and can add an unnaturally heavy load to the small draws leading to the stream unless the road drainage is dispersed into infiltration ditches. Overloading the channels means high velocity flow and bank and bed erosion that put more sediment in movement in the main stream below.

Installations

Clearing the right-of-way may be done in any weather, but use of heavy dirt-moving machinery in opening up the location should be restricted to dry weather. If the machinery is used during wet weather, soil may be compacted or the mud may drain off to streams. Machinery also should be kept off streambanks and out of channels as much as possible. (A "shoo-fly" road on a temporary fill in the Umpqua River, built and used only a month or two in the low-water season, caused heavy sedimentation of the river and damage to aquatic habitat for 25 miles downstream.)

Culvert installations are often made in such fashion that they greatly increase soil erosion. The "cannon" culvert sticking out several feet above a steep fill slope is all too common a sight, as is the long gully below it. Too-short culverts sometimes are used in stream crossings; the fill slopes override the culvert ends to supply sediment to the stream. Buried culverts with unprotected inlets may induce erosion, too. Generally, the solution is to lay the culvert on the natural channel gradient, to make it long enough to extend well beyond the fill slopes, to protect the inlet and outlet with headwalls, and to provide a lined drainageway where a natural drainage route is not available. Drop inlets and protected outfalls will save many cubic yards of erosion sediments.

Drainage collection and disposal may also significantly affect soil loss and sediment contributions. Road ditches should be built so that they do not erode and undermine either the roadbed or the cutbank. Cross-drainage should be by means of structures that will not break down and permit escape of the water to gully out the road. Drainage should not be allowed to concentrate, but should be diverted at short intervals, depending on gradient and soil type. Disposal should be into natural channels of sufficient capacity to handle the maximum expected drainage flow without erosion, into infiltration ditches on contour, or onto areas of undisturbed cover where it can spread out and percolate into the soil. Drainage should never be turned loose to dig its own channel.

Maintenance

The major operations involved in maintenance are regrading the road surfaces to smooth out ruts and redistribute surfacing material, cleaning out soil and rock eroded and sluffed into ditches, and removing slides. Where there is surplus material to take care of, it generally is overcast onto the fill slope. This can nullify the effectiveness of any soil stabilization work done on the fill and may directly contribute to increased erosion and sedimentation.

Amount of maintenance necessary is related to many different factors — road location and design being two of them. Poor location and poor design can cause maintenance costs to rise almost to equal those of construction. Where unstable formations suffer recurrent slips and slides, where poorly placed ditches undercut steep side slopes, where unprotected drainage outfalls gully back into the road, both high sediment contributions and high maintenance costs will be the rule. To lower costs and cut down sediment contributions, maintenance should avoid undercutting the side slopes or plugging drainage ditches and should conserve surfacing material.

Effects

Three specific effects of sediment from roads might be named: (1) damaging the aquatic habitat, (2) degrading water quality, and (3) lowering the attractiveness of streams for recreation. Sedimentation damage to aquatic habitat arises from smothering of fish eggs and fry in stream bottom spawning beds, blanketing and smothering stream bottom plants and animals that make up the food chain, clouding the water and cutting off light from stream organisms, and at times even abrading the gills of adult fish. The damage to water quality arises from the presence of suspended material that prevents use of the water without treatment to remove the unwanted additions; the treatment is costly. Sediment in water decreases the efficiency of purification by chlorination, and increases the cost of even this simple treatment. Attractiveness of streams and lakes for recreation purposes is based on the preference people generally show for clear, clean water, whether for drinking or bathing, or just sitting beside. Turbid, sediment-laden waters are not attractive, are not very good for fishing, and may conceal safety hazards.

Evaluation of these effects in economic terms is difficult and unsatisfactory. Even such an apparently readily defined sediment damage as loss of reservoir storage space is not adequately evaluated in terms of cost of construction of that storage space, if we are concerned that no other storage space may be available. Damage to aquatic habitat may be measured in terms of loss of production of commercial fish; but what of the loss to the sports fisherman? Damage to water quality may be partially evaluated in terms of treatment costs; but what of water uses that perforce must be abandoned because they cannot afford this cost? No one yet has discovered how to evaluate esthetics; we know only that most outdoor recreation activity is tied to water, that we want the water clean and sparkling, and that we feel a loss when we can't find the conditions we like.

Solutions

A number of problem situations causing or affecting sedimentation have been cited, and it has been pointed out repeatedly that avoidance of sediment is the best treatment. It is always easy, after the fact, to say, "You shouldn't have done it in the first place." But, before the fact, there are certain useful tools available.

Informed land management is based on full knowledge of what is being managed, the objectives and main direction of management, and possible side effects. The full knowledge of what is being managed refers to the land itself — the soil, the underlying rock, topography and

land forms, the hydrologic regime, the plant cover, and accessory factors such as climate and land use. This means that soil maps, geologic maps, land form maps, topographic maps, or any other source of information should all be used in laying out the preliminary road location. Since land-form maps are available in few places (in some parts of the State of Washington, in the Northwest), aerial photographs can be used for the same purpose. Even with these aids and with a ground check, it is not possible to miss every unstable area; some will show up only when cut into.

Planning road layout together with planning of the uses and developments it is to serve will enable construction of the most efficient road system for an area, with compromises recognized as such and adequately allowed for. If water quality and aquatic habitat are considered when planning the layout, erosion and sedimentation hazards can be greatly reduced. Although such planning is possible and desirable for the local forest access or mineral access or farm-to-market road, it may not be possible for the interstate expressway. Still, the local needs and local disruptions (including water quality and aquatic habitat) should be considered and any needed compromises in the highway plans made accordingly.

The planning should include the stabilization work needed not only to protect the road itself but also to safeguard the streams and water resources. The stabilization job can be done more efficiently and with better chance of success if it is made a part of the construction job. This stabilization work may include terracing the cut slopes; staking and wattling and mulching both cut and fill slopes; seeding and planting all bare soil areas; installing drop inlets and protected outfalls on culverts; diverting drainage above cuts and away from fills; and cleaning out carefully and completely any material put into streams. The material to be removed would include temporary fills and culverts as well as refuse inadvertently dropped into the streams.

We might summarize the road problems and solutions as follows:

Construction:	*What To Do About It*
Planning	Integrate considerations of road location, design, and use with considerations of all resources and uses affected, including water resources and aquatic habitat in streams.
Location	Avoid unstable soil and rock, choose bench locations, avoid encroachment on or interference with streams, limit cut and fill.
Clearing right-of-way	Preserve low cover for soil protection as long as possible, keep ash and soil and debris away from channels.
Cuts	Provide drainage diversion at top and in the cut as needed, leave support for the toe; fix slope according to local soil and topography; build terrace as needed; stake, mulch, and revegetate.
Fills	Divert drainage away from fills and protect drainage on fills, don't overcast excess material where it will roll or erode into natural channels, stabilize bare soil surfaces to avoid erosion, install toe structures as needed to support the fill, end haul rather than overcast excess material.
Channel relocation	Avoid where possible, or do it on solid rock bottom as far as possible.
Ditches	Cut ditches on regular grade, with sufficient capacity to carry greatest expected flow, and line against erosion; avoid undercutting side slopes.
Drainage turnout	Divert drainage at short intervals according to climate, soil type, and gradient; dispose into natural channels, into contour infiltration ditches, or onto permeable forest floor.
Culverts	Install on natural grade with minimum disturbance of channel and extend well beyond the fill; protect inlet and outlet against erosion; use drop inlets where fill can act as sediment trap or as necessary to prevent erosion; use no "cannon" culverts on steep slopes.
Shaping and surfacing.	Carefully outslope and smooth the surface to reduce need for drainage structures and to keep erosion to minimum, round the surface to drain water to side ditches, apply sufficient rock course to provide a surface free of muddy ruts.
Maintenance:	
Regrading	Redistribute surfacing material without wasting or overcasting, fill ruts and leave berm to contain drainage as needed.
Drainage system	Keep ditches on even gradient and cleared to avoid overflow; do not undercut sides when cleaning ditches; see that clear drainage goes to natural channels where possible and muddy drainage to infiltration areas; keep culverts unplugged.

Slide removal	Rather than overcast excess material, end haul to safe disposal; cut diversion drains above wet cutbanks or insert French drains into the bank; build toe support as needed.
Repair	Fill and restabilize eroded or slumped areas.

Summary

The thousands of miles of new road built each year and the tremendous existing road network being maintained involve the disturbance of hundreds of thousands of acres of land and millions of tons of soil. Much of the disturbed soil erodes and becomes sediment in streams, damaging the aquatic habitat, degrading water quality, and lowering the attractiveness of the streams for recreation use. This is a situation that can be in large measure avoided. From planning the location and design of the road on through to care of the finished product, there are precautions to take; plus a number of positive actions that will enable holding soil disturbance and erosion and sedimentation to a minimum.

SEDIMENTATION AFTER LOGGING ROAD CONSTRUCTION IN A SMALL WESTERN OREGON WATERSHED

[Paper No. 8]

By R. L. FREDRICKSEN, *research forester, Pacific Northwest Forest and Range Experiment Station, Forest Service, Portland, Oreg.*

Abstract

During the summer of 1959, 1.65 miles of logging road were constructed in a 250-acre forested watershed that rises 2,000 feet in a distance of 1 mile. This study evaluates the change in sedimentation subsequent to road construction. Runoff from undisturbed watersheds in this area remains clear during the summer low-flow months and reaches concentrations of 100 parts per million during winter storm peaks. Runoff from the first rainstorms after road construction carried 250 times the concentration carried in an adjacent undisturbed watershed. Two months after construction, sediment had diminished to levels slightly above those measured before construction. Sediment concentrations for the subsequent 2-year period were significantly different from preroad levels. In about 10 percent of the samples, sediment concentrations were far in excess of predicted values, indicating a streambank failure or mass soil movement. Annual bedload volume the first year after construction was significantly greater than the expected yield, but the actual increase was small. A trend toward normalcy was evident the second year.

Introduction

Streams flowing from undisturbed mountain watersheds of western Oregon normally carry very small sediment loads. But when logging roads are built to harvest the old-growth Douglas-fir timber from these watersheds, the construction activities expose considerable raw soil, often resulting in increased sedimentation. In 1952, the Forest Service began a watershed experiment designed to measure the effect of intensive forest land management upon the sediment load carried by streams in the western Cascade Range of Oregon. The first treatment phase began in 1959 when logging roads were built in one experimental watershed. This paper presents an estimate of the change in suspended sediment concentration after construction of these roads.

Colby, Hembre, and Rainwater,[1] in a thorough investigation of the Wind River basin of Wyoming, found annual sediment yield ranged from 1.11 to 0.70 ton per acre during a 5-year period. They found large differences in the sediment load carried from watersheds draining different types of geologic materials. Care must be taken when projecting sedimentation rates from small watershed studies to larger watersheds, particularly where there is a change in geologic material.

Anderson[2] was able to segregate sediment load in the Willamette River basin of western Oregon into three sources: (1) 24 percent from forest lands comprising 77 percent of the drainage area, (2) 22 percent from agricultural land comprising 23 percent of the area, and (3) 54 percent from 205,000 feet of eroding main channel. He predicted that if forest land development continued at the rate existing at the time of the study, sediment discharge would increase to three times the rate that was estimated for the watershed condition in 1950.

The progress of erosion from a small watershed was measured for 3 years after logging in the Sierra Nevada of California. Here Ander-

[1] COLBY, B. R., HEMBRE, C. H., and RAINWATER, F. H. SEDIMENTATION AND CHEMICAL QUALITY OF SURFACE WATER IN THE WIND RIVER BASIN, WYOMING. U.S. Geol. Survey Water-Supply Paper 1373, 336 pp. 1956.

[2] ANDERSON, H. W. SUSPENDED SEDIMENT DISCHARGE AS RELATED TO STREAMFLOW, TOPOGRAPHY, SOIL, AND LAND USE. Amer. Geophys. Union Trans. 35(2): 268-281. 1954.

son and Richards [3] found that, once logging was completed and the area began to recover, mean sediment concentration during high streamflow decreased markedly. During the second and third years after logging, it decreased to about half what it had been the previous year.

The Study

On the H. J. Andrews Experimental Forest, located near Blue River, Oreg., three small, gaged watersheds (fig. 1) have been under study to evaluate the effects of logging on the quantity and quality of runoff.[4] In the 1 mile between the gaging site and the back ridge of the watersheds, the elevation increases from 1,500 to 3,000 feet. Topography is steep and broken with deeply incised stream channels that flow northwesterly. Geologic structures include basaltic-andesite ridges overdeposited with tuffs and breccias. Tuffs and breccias are parent materials for deep, heavy, and highly aggregated soils on benches and at the toe of slopes. Runoff in stream channels is rapid, though surface runoff has never been observed. The soil mantle is very permeable.

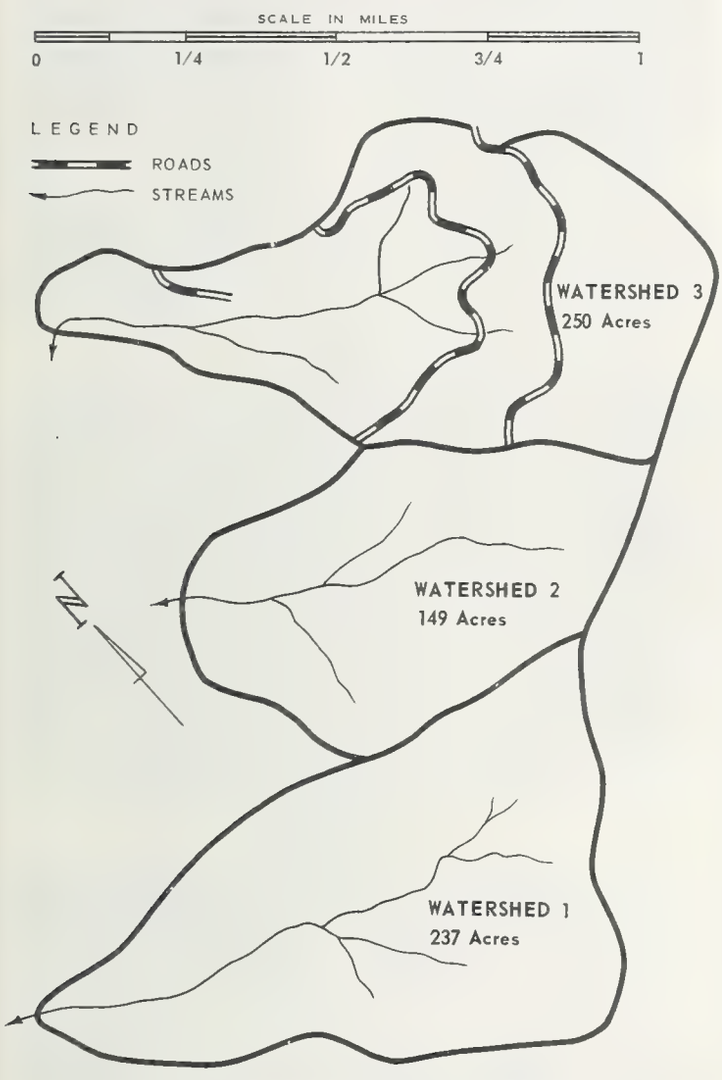

FIGURE 1. — Experimental watersheds.

Cover is predominately overmature Douglas-fir [*Pseudotsuga menziesii* (Mirb.) Franco], varying from 20,000 to 120,000 board feet per acre. The dense cover has remained essentially unbroken for a period of 450 years.

The maritime climate of western Oregon is typically dry in summer and wet in winter. Annual precipitation averages 91 inches but may vary from 56 to 114 inches, with 95 percent of the precipitation falling between October and May. Although large storms with 3 or more inches of precipitation per day may occur during this period, rainfall intensities seldom exceed 0.3 inch per hour. Snow may be present at this elevation from November through March, but only occasionally remains on the ground for more than two weeks at one time.

Road Construction

The experimental watersheds remain undisturbed until the spring of 1959 when construction of logging roads began in watershed 3. By October 1, 1.65 miles of all-weather logging road were completed with a 14-foot roadbed topped by a 10-foot, crushed-rock driving surface. This transportation system consists of three roughly parallel roads at elevations of 1,900, 2,400, and 2,800 feet. (fig. 1). Continuously flowing streams are crossed in two places by the middle road. No surface flow is evident at the lower or upper roads except during major storms.

Annual road maintenance, performed during the summer, consisted of removing several minor slumps along cut banks and clearing drainage ditches. During September 1959, all cut and fill slopes were seeded with grass, fertilized, and mulched with straw, but only a poor stand of grass resulted. No logging trucks used the roads during this phase of the study.

Methods

Beginning in 1955, suspended sediment was sampled at each stream gage (trapezoidal flume). Vertically integrated samples were taken in pint milk bottles from the upstream end of each flume. Results of analysis, by the Gooch filtration technique, are expressed in parts per million (p.p.m.). Bedload has been measured in catchment basins below the gaging sites since 1957. The basins, with 1,650 to 2,050

[3] ANDERSON, H. W., and RICHARDS, L. G. FOURTH PROGRESS REPORT, 1960-61, CALIFORNIA COOPERATIVE SNOW MANAGEMENT RESEARCH. U.S. Forest Serv., Pacific Southwest Forest and Range Expt. Sta. Study 112, pp. 154-155. 1961.

[4] BERNSTEN, C. M., and ROTHACHER, J. A GUIDE TO THE H. J. ANDREWS EXPERIMENTAL FOREST. U.S. Forest Serv. Pacific Northwest Forest and Range Expt. Sta., 21 pp., illus. 1959.

square feet of surface area, have a low trap efficiency. Bedload volume was calculated annually from the mean rise in pond-bottom elevation, measured on intersections of a 3-foot grid.

Results

Annual Sediment Distribution From Undisturbed Watersheds

Distribution of annual sediment concentrations measured in the experimental watersheds follows a pronounced cyclic pattern. Sediment concentration of samples plotted in figure 2 show considerable variation caused by major storms and the short 6-year period of record. But the figure shows clearly that sediment concentrations are small during low runoff summer months and rise in autumn to a peak during high runoff in winter months. Sediment seldom rose above 100 p.p.m. from these undisturbed watersheds, though greater concentrations have been measured. Localized failures of streambanks during storm peaks probably account for the short-lived surges to slightly over 200 p.p.m.

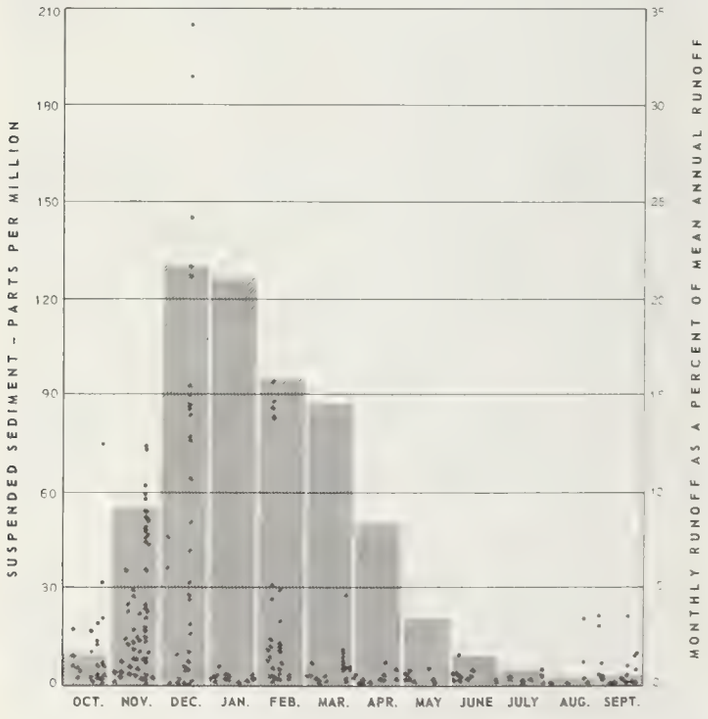

FIGURE 2. — Annual distribution of suspended sediment samples and monthly runoff from undisturbed watersheds, 1957-62.

Changes in Suspended Sediment After Road Construction

A drastic change in sedimentation was apparent immediately following road construction. The increase, measured when the rainy season began in autumn of 1959, compares with results reported by Guy [5] during the construction phase of urban development.

Sediment loads, when streams were near peak flows, are shown as ratios (watershed 3 to watershed 1) in figure 3. Before road construction, peak sediment loads in watershed 3 were 2.3 times those in watershed 1. During the first storm of September 21, 1959, when roads were nearing completion, sediment reached a maximum concentration of 1,780 p.p.m., 250 times the concentration in watershed 1. Two months later, the initial effect of road construction had apparently passed. By November 23, 1959, sediment concentrations in watershed 3 subsided to levels slightly above those measured before construction and remained at about these levels for the following 2 years.

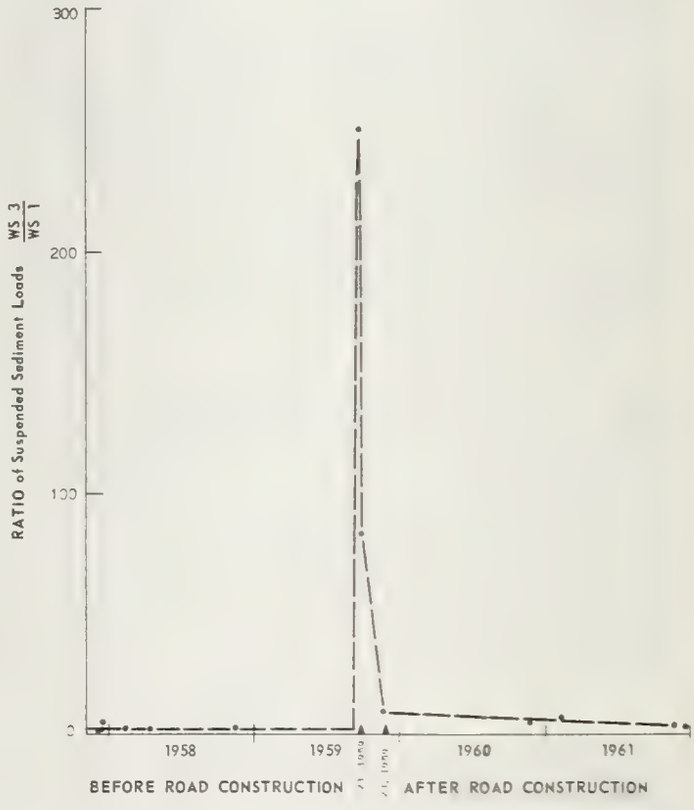

FIGURE 3. — Relative suspended sediment loads near peak flows: Watersheds 1 and 3, before and after road construction.

Data for this 2-year period after road construction were analyzed to determine their relation to data collected before construction. Two regressions of paired samples [6] from watersheds 1 and 3 were calculated and are compared graphically in figure 4. Sediment concentrations in watershed 3 were slightly more than twice

[5] GUY, H. P. EFFECTS OF URBANIZATION ON THE SUPPLY OF FLUVIAL SEDIMENT. U.S. Geol. Survey Res. Prof. Paper 424-A, p. 85. 1961.

[6] Samples were considered paired if both were collected within a 1-hour interval.

the corresponding concentration before construction. This increase proved highly significant.

Slides Add Unpredictable Quantities of Sediment

Relationships shown in figure 4 reflect only "normal" erosion of the soil mantel. The analysis does not include sediment concentrations caused by sudden movements of soil such as slippage of streambanks. In the period of observation, 8 samples out of 83 collected during major storms contained sediment contributed by these unpredictable events and were not included in the analysis.

The largest sediment concentration in watershed 3 during this period was caused by a slide that originated from the middle road and dammed a small tributary. When the dam was breached, a wall of water and debris scoured one-half mile of stream channel to bedrock. Two debris dams containing about 5,000 yards of rock, gravel, and logs were left in the channel. Within 20 hours after the slide, 260 tons of sediment passed the stream gage — many times the yield expected for this period had the slide not occurred.

Bedload

Annual accumulation of bedload material, normally very small, ranged from ¼ to 3½ cubic feet per acre of drainage for the period before road construction. The first year after road construction, bedload was significantly greater than expected (95-percent confidence level), although the actual volume was small during this low runoff year. A trend toward normalcy was evident by the second year. Bedload in significant quantities was deposited in the basin at watershed 3 after the slide described in the previous section. The volume of 10.84 cubic feet per acre was nearly 18 times the volume at undisturbed watershed 2.

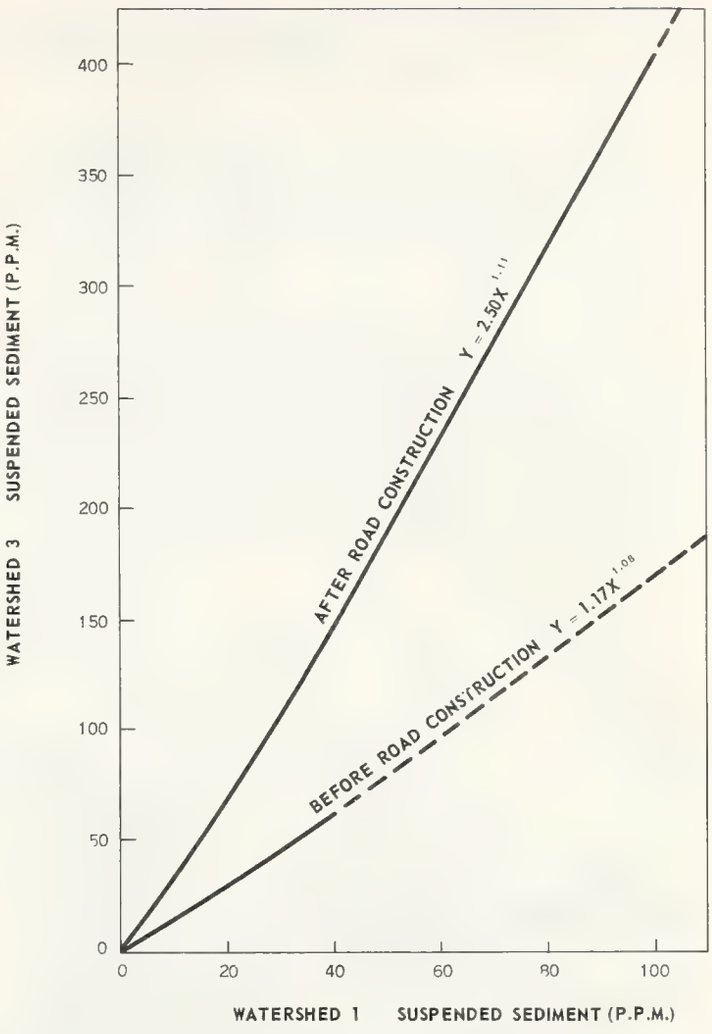

FIGURE 4.—Suspended sediment concentrations: Watersheds 1 and 3, before and after road construction.

Summary

Suspended sediment in undisturbed watersheds follows a cyclic concentration pattern largely influenced by the precipitation and runoff pattern in western Oregon. In these undisturbed watersheds, suspended sediment concentration seldom exceeded 100 p.p.m. Sediment concentrations were much higher for 2 months during the beginning of the 1959 autumn rainy season, immediately after road construction. Sediment concentrations during 1960 and 1961 continued at about twice the concentrations measured before road construction. A modest increase in bedload volume was noted during the first year after construction.

A slide in watershed 3 produced quantities of sediment far in excess of previous years. Slides play an important but unpredictable role in the rate of geologic erosion from this physiographic type.

EFFECTS OF WATERSHED CHARACTERISTICS ON RESERVOIR SEDIMENT DEPOSITION

[Paper No. 9]

By ROGER L. CORINTH, *assistant engineer, Illinois State Water Survey*

Scope

This paper attempts to set out a pattern of understanding of sediment movement in Illinois. Sediment measurements within the watershed and the reservoir are combined with knowledge of physical laws to gain understanding of hap-

penings between the sediment-producing areas of a watershed and the deposition in the reservoir. In this "between" zone little or no data are available.

In Illinois, the terrain is flat and cultivated. The soil types range from moderately permeable soils developed from thick loess in the northwest quadrant of the State to very slowly permeable soils developed from thin loess in the southern half.

For the State of Illinois we have 126 reservoir sedimentation surveys, including 1,697 sediment samples, and complete gross erosion computations for 38 watersheds. The reservoirs range from 3 to 6,000 acres, with watersheds from ½ to 906 square miles. The percentages of clay (<2 microns), silt (2 to 30 microns), and sand (plus organic content) in sediments of these Illinois reservoirs are shown in figure 1 for three principal study areas in the State.

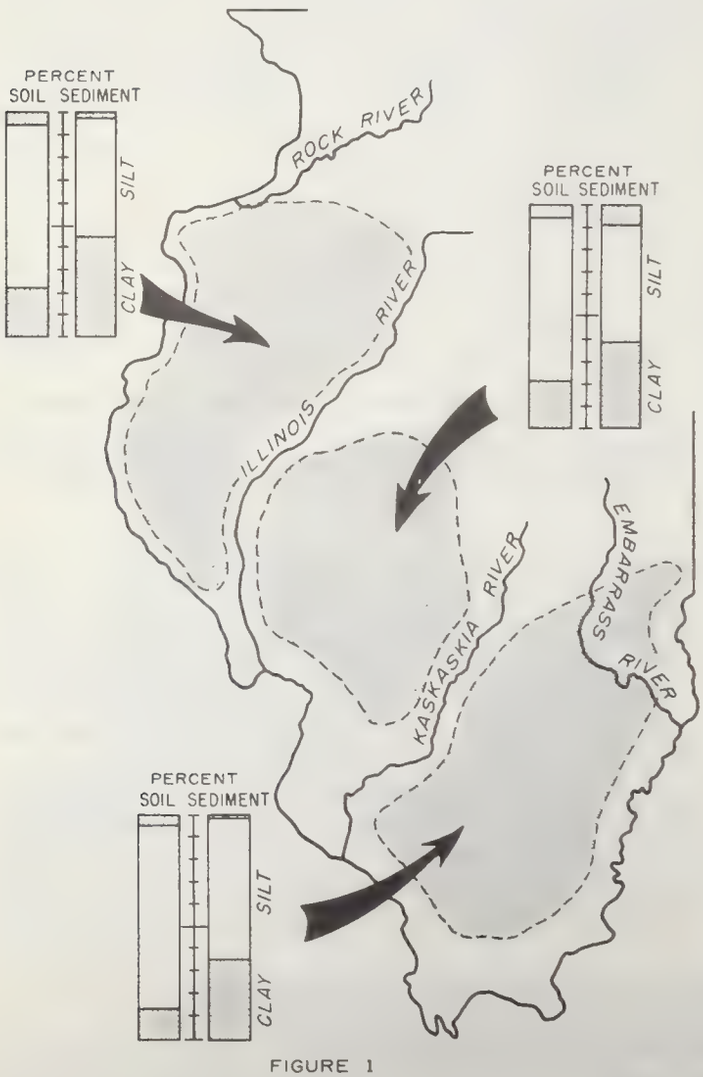

FIGURE 1

FIGURE 1. — Silt and clay content of sediment in Illinois reservoirs.

Principles of Sediment Movement

Soil erosion and the resulting reservoir sediment deposition are the composite result of a limited number of sequential processes out of many erosion and deposition processes, large and small. For the purpose of focusing our attention on reservoir sediment deposition, let's first clearly identify the elements of the problem and their interrelationship.

The problem has its beginning with the mechanism of soil erosion. Explanation of the movement of a unit mass as a process in soil erosion requires finding a source of energy. This obviously is an example of Newton's first law, "bodies at rest remain at rest until acted upon by some force." In most cases only one source of energy is considered dominant. In the Corn Belt States, the dominant energy source is derived from rainfall.

Three Sediment Movement Regimes

Regimes 1 and 2

From the definition of energy of a body as the amount of work it can do by virtue of its position or motion, the following two erosion regimes become apparent: (1) erosion resulting from kinetic energy [1] and (2) erosion resulting from the relationship of momentum and hydraulics. Regime 1 is a function of turbulence resulting from expenditure of the kinetic energy of rainfall and the turbulence inherent to open channel flow. The areal extent of this regime is a function of turbulence and depth of runoff.

The line separating regimes 1 and 2 is located where the runoff depth becomes such that the extreme turbulent energy per unit volume due to raindrop penetration does not reach through the depth of runoff, thus giving way to open channel flow with a far lesser amount of turbulent energy per unit volume. Beyond this point the turbulence characteristic of open channel flow minus the turbulence of the raindrop penetration must maintain the sediment load. This point of separation must be located in the search for watershed characteristics. This point may reach well up into the cultivated fields as opposed to its location in watersheds protected by cover such as forest.

The eroded soil carried as suspended load in the vicinity of the periphery of a cultivated field will very nearly appear in total at the reservoir head waters. Watersheds characterized by ephemeral streams appear quite stable, with little evidence of active degradation or aggradation. This means either that there is little or no

[1] WISCHMEIER, W. H., and SMITH, D. D. RAINFALL ENERGY AND ITS RELATIONSHIP TO SOIL LOSS. Amer. Geophys. Union Trans. 39: 285-291. 1958.

deposition in the watershed channels or that aggradation and degradation average out to a state of equilibrium. That is, sediment is alternately perched and transported with a like quantity replacing the unperched quantity. In either case the attenuation of watershed erosion (computed gross erosion by a soil loss equation)[2] caused by deposition in watershed channels is not supported by evidence, here in Illinois, of continued channel aggradation.

This precludes the consideration of regime 2 as a significant factor in attenuating gross erosion and suggests that the area of significant effect is in the zone of transition between regimes 1 and 2. However, regime 2 would be important where the overbank flooding produced by the larger storms gives rise to broad areas of temporary storage where considerable deposition may take place. This overbank storage would attenuate the total sediment load produced by the large storms. The inference here is that smaller watersheds are not likely to contain channels in broad U-shaped valleys, whereas large watersheds do contain broad U-shaped valleys. The same characteristics that differentiate a sharp-peaked, flashy hydrograph from one that is not would also attenuate the sediment load being transported through regime 2.

Supporting evidence of the effect of this potential overbank deposition is demonstrated by graphs of sediment production per acre of watershed vs. watershed area, which shows a general decrease in sediment production with increasing watershed size.

Support of regime 1 as being a dominant sediment source is borne out by the estimate that 90 percent of reservoir sediment in the United States is contributed by sheet erosion.[3] Also, in only 1 out of 38 Illinois watershed soil loss computations was channel erosion a significant percentage of the total erosion. Then, by default, the existence of regime 2 must be in the form of a nonsediment producing medium or as a sediment transport and deposition medium in equilibrium.

Regime 3

Regime 3 embodies the reservoir proper. Here the principles of fluid mechanics and Stokes law govern in relating sediment inflow to deposition. This completes the three divisions (1, 2, 3) shown in figure 2 representing the three regimes of sediment movement.

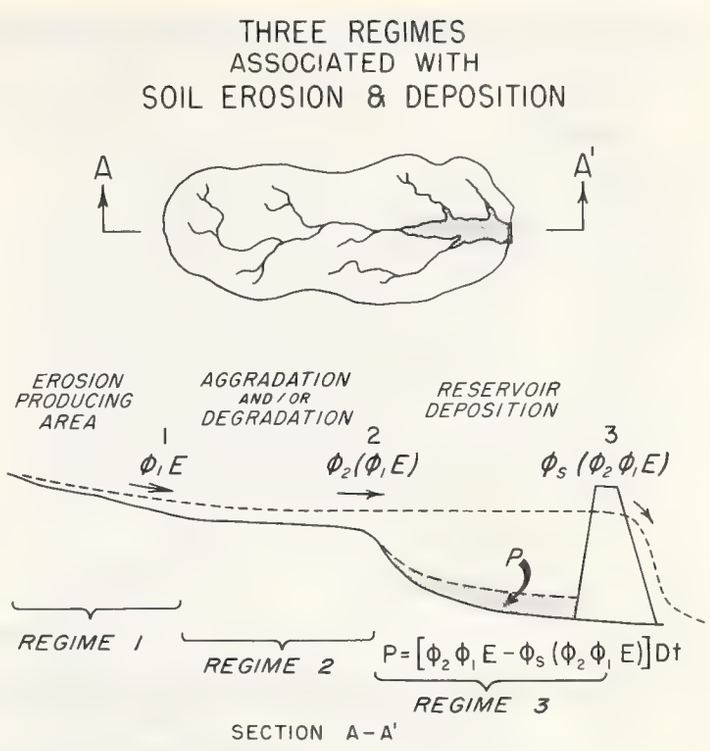

FIGURE 2. — Three regimes associated with soil erosion and deposition.

We next place some qualitative symbols on the three regimes as follows:

P = tons, reservoir deposition;
E = tons per acre-year, computed gross erosion;
D = acres, watershed area;
t = years;
ϕ_1 = coefficient of sediment discharge from regime 1;
ϕ_2 = coefficient of sediment discharge from regime 2;
ϕ_s = coefficient of sediment discharge from regime 3.

With reference to figure 2, the relationship of these mentioned seven variables in a sediment budget equation is as follows:

$$\phi_2 (\phi_1 E)\, Dt = P + \phi_s (\phi_2 \phi_1 E)\, Dt \quad (1)$$

$$P = \phi_2 E - \phi_s (\phi_2 \phi_1 E)\, Dt \quad (2)$$

$$P = \phi_2 \phi_1 E (1 - \phi_s)\, Dt \quad (3)$$

An overall pattern of understanding has now been laid out for sediment movement throughout the watershed and reservoir.

Physical Evaluation of Regimes 1, 2, 3

In the study of watersheds, assessing the sheet erosion contribution to sediment load has been very successful by evaluating on a watershed basis the universal soil loss equation.

Many recent authoritative articles have attacked the problem of isolating watershed characteristics that attenuate computed gross erosion to the extent that a quantitative prediction of

[2] VAN DOREN, C. A., and BARTELLI, L. J. A METHOD OF FORECASTING SOIL LOSS. Agr. Engin. 37(5): 335-341. 1956.

[3] GLYMPH, L. M., JR. RELATION OF SEDIMENTATION TO ACCELERATED EROSION IN THE MISSOURI RIVER BASIN. U.S. Soil Conservation Serv. SCS-TP-102. 1951. [Processed.]

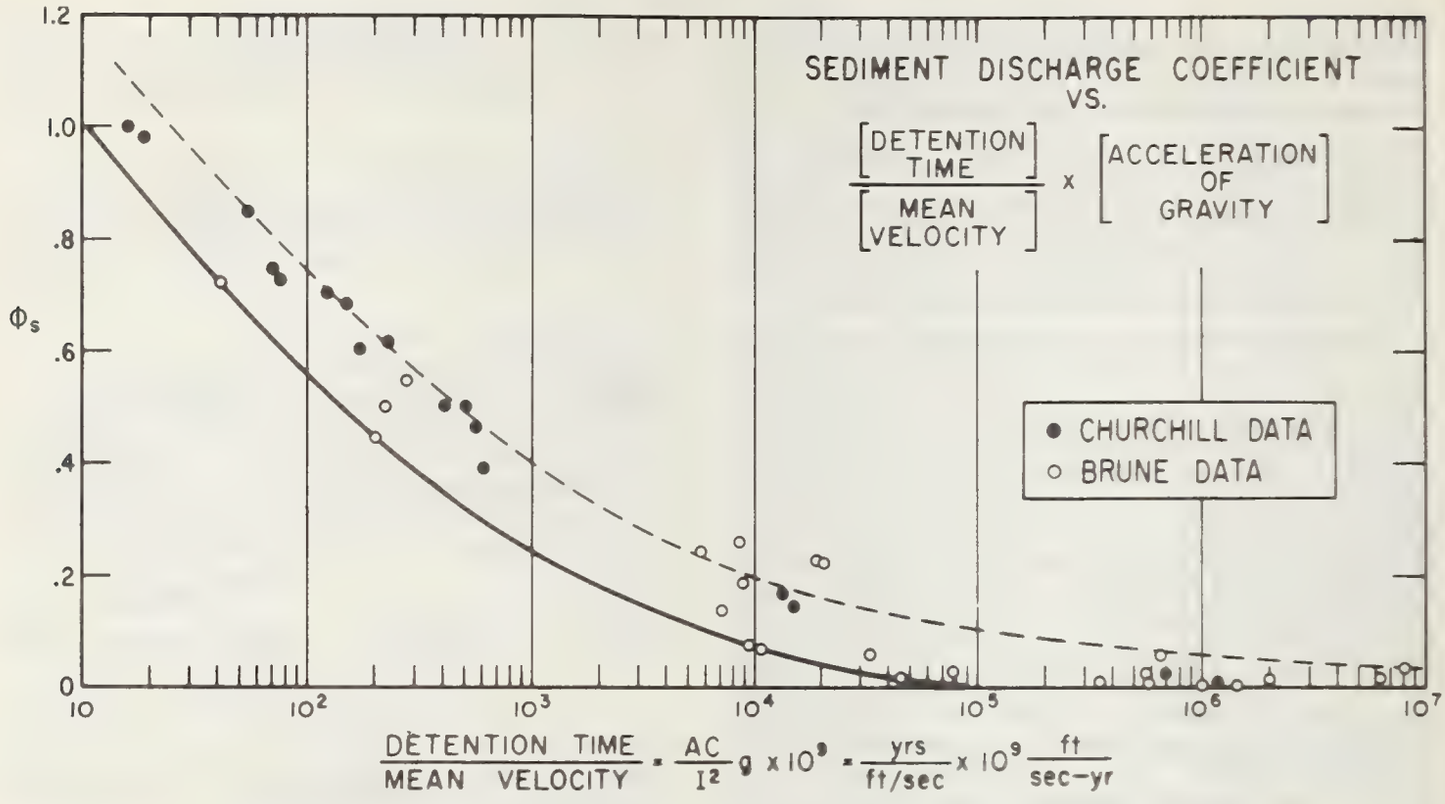

FIGURE 3. — Sediment discharge coefficient.

reservoir deposition is possible. In general, these studies have been on a macro scale in the watershed, thus encompassing regimes 1 and 2. Considerable success has been achieved under certain conditions, as in the case of Maner [4] who found, in the Rolling Red Plains area in Texas, a strong relationship between a relief-length ratio and sediment delivery rate.

Evaluation of regime 3 has commonly been by volumetric measurement of sediment deposited and in some cases by suspended load measurements at points 2 and 3 in figure 2.

Analysis of Interrelationship of Regimes 1, 2, 3

The area of primary interest to us in this system of regimes is the volume loss due to reservoir sedimentation. Thus, our program is oriented toward a reliable prediction of P in equation 3.

Of the seven variables in equation 3, P, D, and t are readily obtainable, whereas E is a computed value. The term $(1 - \phi_s)$ corresponds to the commonly known reservoir trap efficiency relationships.[5]

Reservoir and river sedimentation is commonly a function of one-directional flow. Therefore, trap efficiency in terms of $(1 - \phi_s)$ affords perhaps a more logical explanation of the extremes in trap efficiency. In the particle size range associated with waterborne sediments, ϕ_s, the coefficient of sediment discharge from regime 3 would theoretically never be zero. At the other extreme, a ϕ_s value in the vicinity of 1.0 would be at the threshold of scouring conditions. A continuity in the expression $(1 - \phi_s)$ exists as reservoir size increases from impoundments created by submerged dams and channel obstructions to structures of large C/W ratios.

Data have been presented in a form corresponding to this relationship by Churchill,[6] who relates the sediment passing through a reservoir to a sedimentation index in units of the reciprocal of acceleration. For the reservoirs appearing in Brune's "Trap Efficiency of Reservoirs," detention time divided by mean velocity has been computed and plotted as shown in figure 3.

The introduction of acceleration of gravity

[4] MANER, S. B. A METHOD FOR ESTIMATING SEDIMENT YIELD IN THE ROLLING RED PLAINS LAND RESOURCE AREA. U.S. Soil Conservation Service, Fort Worth, Tex. March 1957. [Processed.]

[5] BRUNE, G. M. TRAP EFFICIENCY OF RESERVOIRS. Amer. Geophys. Union Trans. 34: 407-418. 1953.

[6] CHURCHILL, M. A. ANALYSIS AND USE OF RESERVOIR SEDIMENTATION DATA. Inter-Agency Sedimentation Conf. Proc. Washington, D. C. Discussion by L. C. Gottschalk, pp. 139-140. 1948.

makes the expression in figure 3 truly dimensionless as follows:

$$\frac{AC}{I^2} \times \frac{g}{1} = \frac{L^2 \times L^3}{(L^3/T^2)} \times \frac{L}{T^2} = \frac{L^6\,T^2}{L^6\,T^2}$$

This permits an equation for the curve to be written as:

$$\phi_s = f\,(\frac{AC}{I^2}\,g)$$

where

A = reservoir surface area;
C = reservoir capacity;
I = rate of average annual inflow;
g = gravitational acceleration;
ϕ_s = sediment discharge coefficient;
L = length;
T = time.

Accepting the sediment discharge coefficient relationship exemplified in figure 3 now makes the problem of interrelating regimes 1, 2, 3 determinant in that there remains only one unknown, the expression containing $\phi_2\ \phi_1$.

Thus, as is often done, the constants and knowns are divided out, leaving the unknown expression for sediment discharge from regime 2 on the right side of the equation; however, in accord with the preceding arguments, the composite unknown $\phi_2\ \phi_1$ remains on the right side as follows:

$$\frac{P}{D\,t\,E\,(1-\phi_s)} = f\,(\phi_2\,\phi_1) \qquad (5)$$

The left side is the dependent variable in evaluating watershed characteristics that are exemplified by $f\ (\phi_2\ \phi_1)$.

In our approach to the data contained in this equation we make certain necessary assumptions as to the rates of change for these factors on the left, namely, that their rates of change are negligible. However, as the quantity of data builds up, we will in time approach the realm of evaluating their rates of change.

Of more immediate value is a consideration of the variation of ϕ_s with time. Reservoir capacity and ϕ_s, by comparison with all other factors at the present level of evaluation, have the most significant change with time. Plotting P against t from equation 3 gives a curve of the general shape shown in figure 4, which is a mass curve of sediment accumulation. The first derivative of equation 3 with respect to time then is interpreted as the instantaneous rate of change of the mass curve of P, or:

$$\frac{dP}{dt} = \phi_2\ \phi_1\ E\ (1-\phi_s)\ D \qquad (6)$$

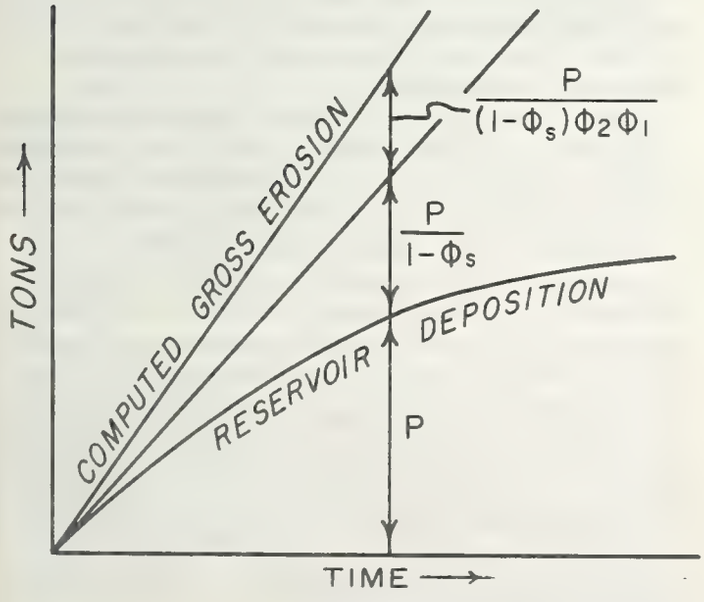

FIGURE 4. — Mass curves of erosion and deposition.

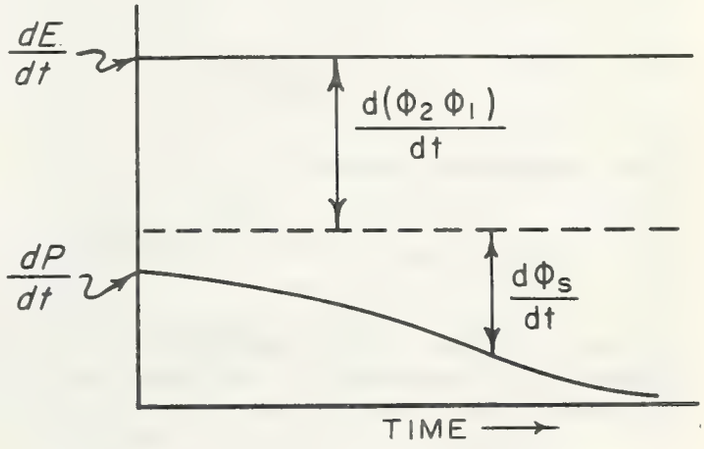

FIGURE 5. — Time derivative relationships.

Figure 5 shows the relationship of $\frac{dP}{dt}$ with time. Shown also is $\frac{dE}{dt}$, which through necessity is assumed to be constant. The horizontal dashed line represents $\frac{dP}{dt}$ corrected for trap efficiency, $(1-\phi_s)$. The ordinate then between $\frac{dP}{dt}$ and the dashed line is $\frac{d\,\phi_s}{dt}$, whereas the ordinate between the dashed line and $\frac{dE}{dt}$ remains to be explained by means of watershed characteristics. Figure 6 shows the relationship of ϕ_s with time.

Consider now the reverse case, where the mass curve of P is to be predicted for a proposed reservoir. The ordinate to the mass curve is now equal to the area under the $\frac{dP}{dt}$ vs. time curve shown again in figure 7 and is expressed as follows:

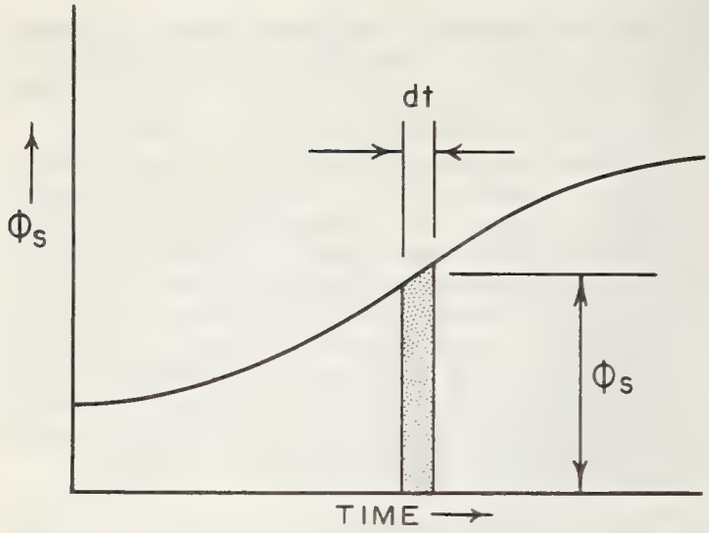

FIGURE 6. — Sediment discharge coefficient relationship.

$$P = \int_0^t \left(\frac{dP}{dt}\right) dt \qquad (7)$$

If we substitute equation 6,

$$P = \int_0^t [(\phi_2\ \phi_1\ E)\ (1-\phi_s)\ D]\ dt$$

The only variable we are presently capable of evaluating with respect to time is ϕ_s, and with all others being constant, then:

$$P = \phi_2\ \phi_1\ E\ D \int_0^t (1-\phi_s)\ dt \qquad (8)$$

$$P = \phi_2\ \phi_1\ E\ D \left[\int_0^t dt - \int_0^t \phi_s\ dt \right]$$

$$P = \phi_2\ \phi_1\ E\ D \left[t - \int_0^t \phi_s\ dt \right] \qquad (9)$$

The value of the integral in equation 9 is found to be the area under the ϕ_s vs. time curve shown in figure 6, and will be important in considering the life expectancy of a reservoir.

In data collection, the integral factor could be important where sedimentation surveys are made at a time in a reservoir's life, when the ϕ_s value is in the vicinity of rapid change. However, if conditions are such that ϕ_s may be assumed constant, equation 9 simply reverts to the form of equation 3, since the integral would then be $\phi_s\ dt = \phi_s\ t$.

When the effect of time on ϕ_s is considered in the analysis of watershed characteristics, equation 5 becomes:

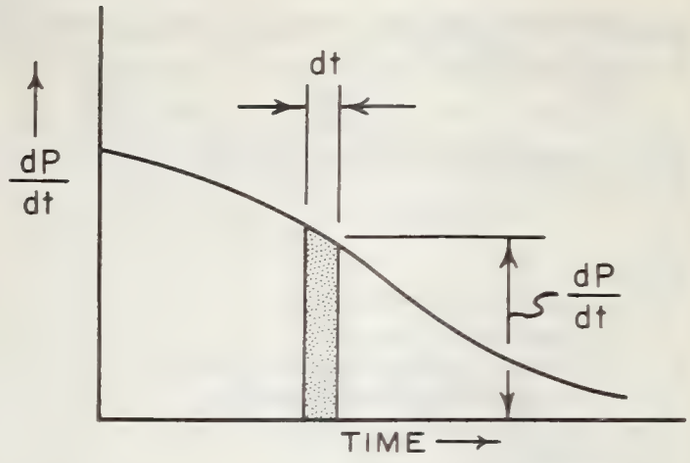

FIGURE 7. — Annual sediment deposition relationship.

$$\frac{P}{D\ E\left[t - \int_0^t \phi_s\ dt\right]} = f\ (\phi_2\ \phi_1) \qquad (10)$$

Conclusions

The problem of analyzing the watershed and reservoir as a unit is simplified by considering (1) the attenuation of gross erosion by factors on a macro scale that bridges regimes 1 and 2, and (2) the trap efficiency when it is analyzed as a constant with respect to time.

By the arguments set forth herein, the existence of regimes 1 and 2 is validated, giving rise to the concentration of effort on the evaluation of the zone of transition between regimes 1 and 2. Also the methodology for the evaluation of trap efficiency in its relationship to the sediment budget equation has been presented, which can be important in analyzing data and in designing reservoirs.

Thus, our approach to the interrelationship of the three regimes should use to the greatest extent possible our body of physical relations known to govern, and our findings should be in accord with other supporting evidence.

Advances on the zone of complexity, exemplified by $\phi_2\ \phi_1$, have been by way of making measurements of individual watershed characteristics. The pattern of understanding described in this paper is based largely on physical laws and is presented with the hope that it will allow all available reservoir and watershed information to be used to understand better the happenings in the void area between known watershed soil loss and known reservoir sediment deposition.

BUILDING A NONLINEAR SEDIMENT YIELD MODEL

[Paper No. 10]

By ROGER P. BETSON, *head, Statistical Analysis Unit, Hydrology Section, Hydraulic Data Branch, Division of Water Control Planning, Tennessee Valley Authority*

Abstract

Several examples are presented, illustrating the development and solution of a complex empirical sediment yield model utilizing nonlinear least squares techniques adapted for computer analysis.

Introduction

The suspended sediment transported from a watershed is a measure of the erodibility of the watershed. Changes in the yield of suspended sediment can provide a relatively sensitive index with which the effects of land use changes or changes in levels of land management can be evaluated.

The yield of suspended sediment, however, is a complex function of many hydrologic variables as well as the physical characteristics of the watershed. Large variations that are inherent in suspended sediment data tend to obscure any changes in the yield that may have resulted from land use changes. In order to make any quantitative analysis of sediment data, it is usually necessary to make some adjustment for the more important factors causing variation. Because of the large number of factors at work simultaneously, this adjustment is frequently achieved through multivariable regression analysis.

Prior to the advent of high speed electronic computers, devising mathematical sediment yield models and evaluating these models by regression analysis were tedious. The labor involved in analyzing data and solving a regression equation with a desk calculator precluded a trial and error approach to model building. The model itself, of necessity, had to be elementary and preferably with linear regression coefficients. Once the form of the equation was decided and the equation fitted to the data, extensive manipulation of the equation based on the results of the fitting were difficult to justify because of the expense and time involved.

Computers have changed this picture considerably. Computer programs are now available, or can be written, that will evaluate almost any equation that can be devised for data adjustment. This paper illustrates how a computer can be used in the analysis of a complex suspended sediment yield equation and how the model can be manipulated to yield certain desired results. The basic equation that is used in this study has been presented earlier.[1] No attempt has been made to drastically alter this basic equation by introducing different variables; nor has new data been introduced, although subsequent measurements have been made.

[1] COOPER, A. J., and SNYDER, W. M. EVALUATING THE EFFECTS OF LAND-USE CHANGES ON SEDIMENT LOAD. Amer. Soc. Civil Engin. Proc. Paper 883. 1956.

The Watershed

The Tennessee Valley Authority, as part of its regional resource development program, has recognized the necessity of giving appropriate attention to the solution of land use and water control problems on rural watersheds within the Tennessee Valley. Watershed projects have been included in the program to study the land use and land management problems. Some of these watershed projects are designed to promote basic hydrologic research, whereas others are concerned with demonstrating the effect of TVA's applicable regional development integrated on a single watershed. Hydrologic data on runoff and sediment loads are collected that serve as measures for determining the effect of changes in cover and land use. In the Chestuee Creek watershed, used for this study, TVA and other Federal, State, and local agencies worked with the people of the area in a program designed to promote efficient control and utilization of water and improved land use, including reduction of erosion and loss of valuable soil.

The Chestuee Creek watershed includes an area of 133 square miles located in the Great Valley of East Tennessee about midway between Knoxville and Chattanooga. Chestuee Creek is one of several streams that run in a northeast-southwest direction in this part of the Tennessee Valley, draining into the Hiwassee River arm of the Chickamauga Reservoir. In an 18-year period, 1944 through 1961, during which hydrologic measurements were collected on this watershed, annual precipitation ranged from 41.0 to 62.4 inches, averaging 52.8 inches, and annual runoff ranged from 14.2 to 28.3 inches, averaging 21.3 inches.

The stream pattern consists of the main Chestuee Creek and a nearly paralled tributary, Middle Creek, in the upper end of the watershed. Smaller creeks and branches drain the higher lands into Chestuee and Middle Creeks. Progressive deterioration of land cover on this watershed in the past century, with consequent accelerated erosion, has resulted in partial filling of the stream channels with resultant reduction in the floodwater carrying capacities.

Extensive hydrologic measurements of precipitation and streamflow were made over the

watershed during the period 1944 through 1961. Sediment measurements were made from 1944 through June 1955 and in the calendar year 1961. Considerable emphasis was placed in this early investigative work on monthly sediment load data because it was felt that this provided one of the better indices with which to measure the effects of improved cover and land use management on the hydrology of the watershed. All the sediment yield analyses were performed on data collected during the 10-year period, 1944–53. The data collected during the 18 months from January 1954 through June 1955 and calendar year 1961 were reserved for a check on the validity of the results. All the studies made in this paper were performed on this same 10-year data so that comparisons of results obtained using various techniques could be made. Studies are also limited to Chestuee Creek at Dentville, the largest gaged area in the watershed, totaling 114 square miles.

Estimating Sediment Yield

The original equation that was used to adjust the Chestuee Creek data was empirical, as are all of the subsequent modifications that were tried for this paper. All the equations studied for this paper, however, were the result of modifications of the original, which included, as the most important variables, current rainfall, antecedent rainfall, and a condition of the cover or season. The use of monthly data limited the available variables to those that could be given meaning over the span of a month. Also included in the eqaution was a time variable to enable the relationship to reflect changes in land use. The form of the equation was as follows:

$$Y = a + b\,P_1^m\,P_2^n + c\,P_1^m\,P_2^n(2 + \sin M) + de^{-fT}$$

where:

Y = monthly sediment, in tons per square mile;
P_1 = rainfall for current month, in inches;
P_2 = rainfall for previous month, in inches;
M = season, by months evaluated by arbitrarily setting January = 30°, February = 60°, etc.;
T = time, by months (January 1944 = 0.01, February 1944 = 0.02, etc.);
e = base of natural logarithms; and
$a, b, c, d, f, m,$ and n are constants to be evaluated.

Solution of this nonlinear equation directly by the use of only desk calculator techniques was virtually impossible, as the equation cannot be easily linearized. Therefore, the equation had to be simplified to reduce the exponents so that linear techniques could be used. Determination of the exponents of the rainfall terms in this equation was done by examining parts of the data in which the effects of the other variables were small. For example, the effect of rainfall on sediment load was studied for similar seasons of the year that also had similar antecedent conditions. The smallest simple integral root or power that seemed to express adequately the effect of that variable on the sediment load was chosen as the exponent of that variable.

Use of the exponential function as an additive parameter in the equation presented considerable difficulty, since with this term included the equation still could not be evaluated directly by the method of least squares. Consequently, an abbreviated form of the equation was used to evaluate the exponent on a trial and error basis. In this abbreviated form the first term of the equation after the constant was dropped to reduce the number of simultaneous equations that had to be solved. In order to evaluate the best value of the coefficient f, various values were assumed. The abbreviated form of the equation was then fitted to data by least squares, and the residual error of the regression computed. The trial values of f were plotted against the respective residual errors. The process was continued until the value of f, which gave the minimum residual error, could be determined from the plot. With the data used in this study, six separate solutions were necessary. After determination of the best value of the exponent f, the equation was fitted to the data by least squares to determine the best values of the other coefficients. Equation 1 shows the results obtained from this fitting.

$$Y = -8.696 - 0.03256\,P_1^2\,P_2^{1/3} + 0.4329\,P_1^2\,P_2^{1/3}\,(2 + \sin M) + 29.88e^{-2.800T} \quad (1)$$

Figure 1 shows the computed time regression for the average monthly sediment load, computed by assigning values to the two hydrologic terms corresponding to their mean. In the 10-year period since 1944, a reduction in the average monthly sediment load of 48 percent is indicated.

Solution of Equation by Computers

Earlier work in TVA has shown the feasibility of using the multivariate technique of component analysis in the solution of nonlinear least squares.[2] This technique for solution of nonlinear equations is found under the name "method of differential corrections." Whenever this technique has been tried in TVA,[3] using multivariate analysis for the solution, reason-

[2] SNYDER, W. M. SOME POSSIBILITIES FOR MULTIVARIATE ANALYSIS IN HYDROLOGIC STUDIES. Jour. Geog. Res. 67(2). 1962.

[3] NIELSEN, K. J. METHODS IN NUMERICAL ANALYSIS. pp. 309-313, New York. 1957.

PLATE 1

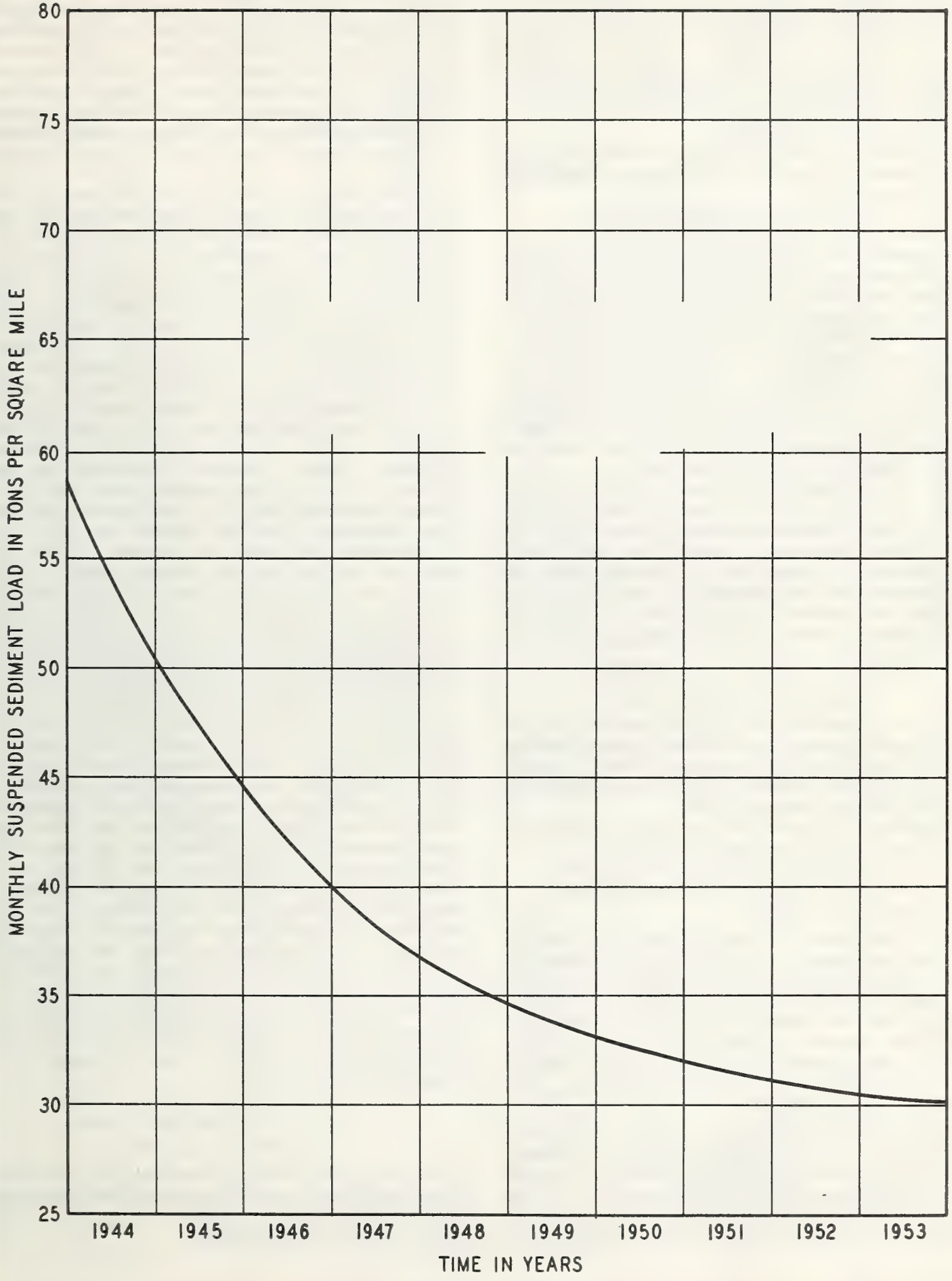

FIGURE 1. — Chestuee Creek Watershed: Average monthly sediment load — regression with time (equation 1).

ably rapid convergence of this iterative solution has been achieved consistently. All the equations that were developed for this paper were solved by this technique with an available master IBM 704 computer program. Operation of this program involves writing a short subroutine containing the partial derivatives of the equation to be solved.

Equation 2 shows the results obtained using the computer to solve for the same coefficients as in equation 1.

$$Y = -9.086 + 0.01625\,P_1^{2}\,P_2^{1/3} + 0.4204\,P_1^{2}\,P_2^{1/3}\,(2 + \sin M) + 30.77e^{-2.832T} \quad (2)$$

The comparability of the coefficients in equation 2 and equation 1 shows that the computer solution accomplished by simultaneous solution for all the coefficients supports the earlier solution achieved by manual trial-and-error regression analysis. The degree of comparability between these two equations is indicated by a comparison of the time regression functions as shown in figure 2. The difference between the amount of change indicated by equation 2, as compared with equation 1, is only 3 percent.

If the computer program is to be of any value, however, the real test is not its ability to reproduce results obtained by the use of desk calculator techniques; rather the test should determine its versatility in solving equations too complex to be attempted by hand solutions. To test the range of the program the sediment yield equation was revised slightly to the following form:

$$Y = a + b\,P_1^{m}\,P_2^{n} + P_1^{m}\,P_2^{n}\,(c \sin M + g \cos M) + de^{-fT}$$

In this equation all of the exponents as well as the coefficients are solved for by the program. In addition, the effect of using a sine + cosine term is to allow variation in both amplitude and phasing of the seasonal wave.

Equation 3 shows the results of fitting this equation to the Chestuee Creek data.

$$Y = -8.988 + 1.054\,P_1^{1.803}\,P_2^{0.4377} + P_1^{1.803}\,P_2^{0.4377}\,(0.5215 \sin M + 0.170 \cos M) + 31.90e^{-3.427T} \quad (3)$$

Again, the values of the coefficients, as well as the two exponents of rainfall, are in substantial agreement with the results obtained in equation 1. Because of the revision in form, the coefficients of the two terms that include the rainfall variables cannot be compared directly. However, if the rainfall variables are factored out of the two terms in which they appear, comparisons can be made.

Figure 3 shows that the variation of the seasonal terms for equations 1 and 3 throughout the year is similar. The two curves are phased within about one-half month of each other. The amplitude of the two seasonal curves differ by about 25 percent between the winter maximum and the fall minimum. The time-regression function, resulting when all of the coefficients and exponent terms in equation 3 are determined by the data, is shown in figure 2. This curve indicates that the suspended sediment transport rate in the watershed was reduced somewhat earlier in the period and also the total amount of the change during the 10-year period is about 9 percent larger than that indicated by equation 1.

One of the more tangible advantages of computer analysis of data is that "hindsight" can be used in the improving of a model to delineate adequately anticipated results. If a model proves inadequate, revisions can readily be made based on the results obtained from initial trials. Often the initial fittings of a model to actual data yield unexpected results, or, on the other hand, the results may be sufficiently encouraging that some sophistication in the model can be justified. An example of this latter case follows.

The time-regression function obtained fitting equation 1 to the Chestuee Creek data yielded desirable results; however, interpretation was difficult. If the two hydrologic terms are given values corresponding to their mean and the time-regression function allowed to vary over 120 months, the equation indicates a monthly reduction of 29 tons per square mile or about 48 percent of the initial rate. This reduction, of course, is an average, and a plotting of the prediction residual error with time would show a characteristic scatter with some months predicted high, others low. In general, the equation underpredicts during winter months and overpredicts during summer months. This seasonal distribution of the error pattern indicates that the change in sediment transport is not the same for all seasons. The time-regression function, in fact, indicates a reduction in the monthly sediment load that is larger than the total load experienced during many of the summer months, even early in the record. It is evident, then, that the amount of the change in sediment transport also varies with the season. Understandably, larger reductions can be expected during the winter months, when the sediment transport is high, than during the summer months. The model as originally constructed, however, allowed variation with time but not by season. To rectify this shortcoming, the following equation, an adaptation of equation 1, was fitted to the data:

$$Y = a + b\,P_1^{2}\,P_2^{1.3} + c\,P_1^{2}\,P_2^{1/3}\,(2 + \sin M) + de^{-fT} + ge^{-hT}\,(2 + \sin M)$$

In this form, the equation allows some seasonal variation in the time-regression function. The result obtained fitting this equation to the data is shown in equation 4.

PLATE 2

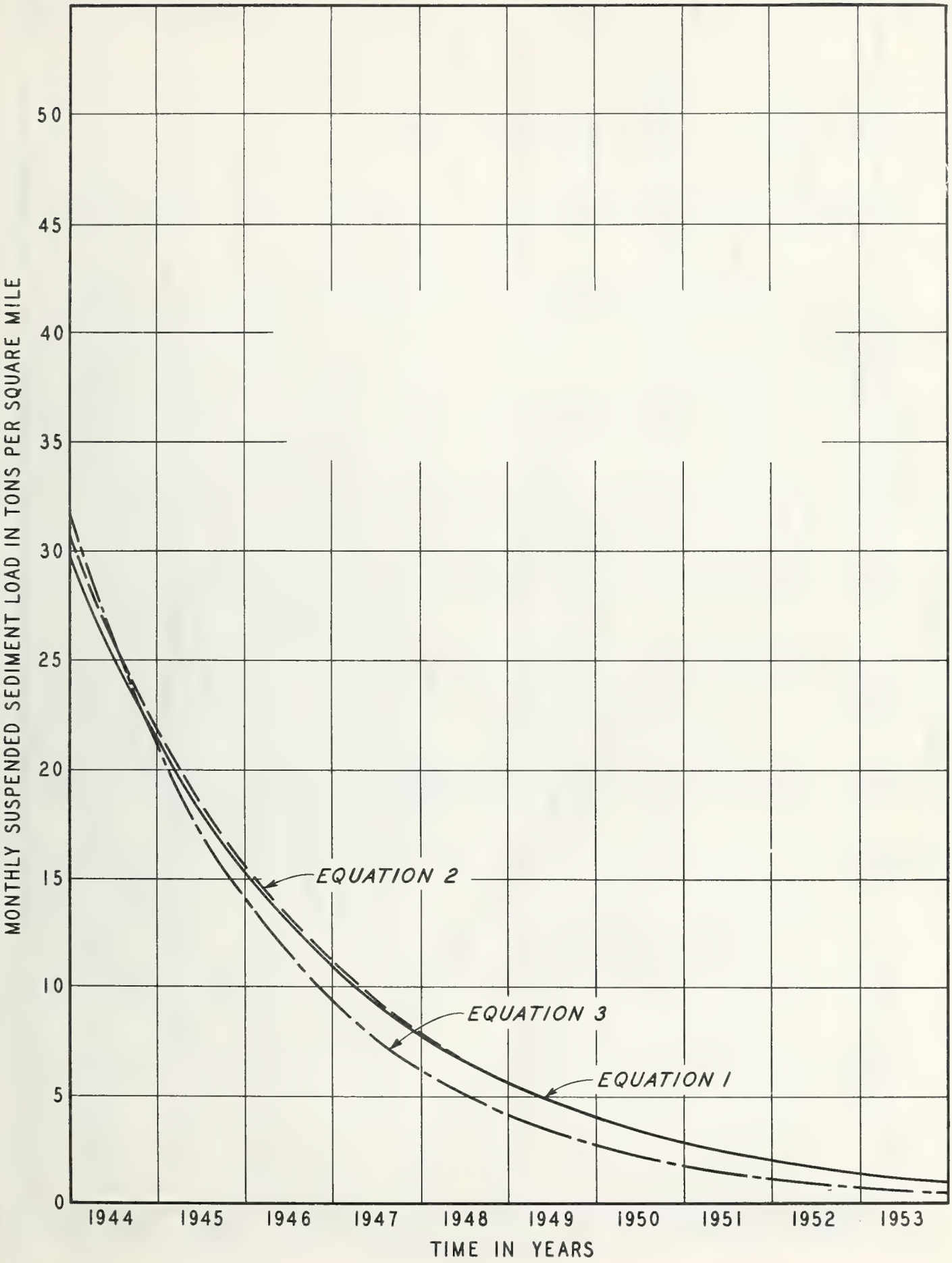

FIGURE 2. — Chestuee Creek Watershed: Suspended sediment prediction by time regression function.

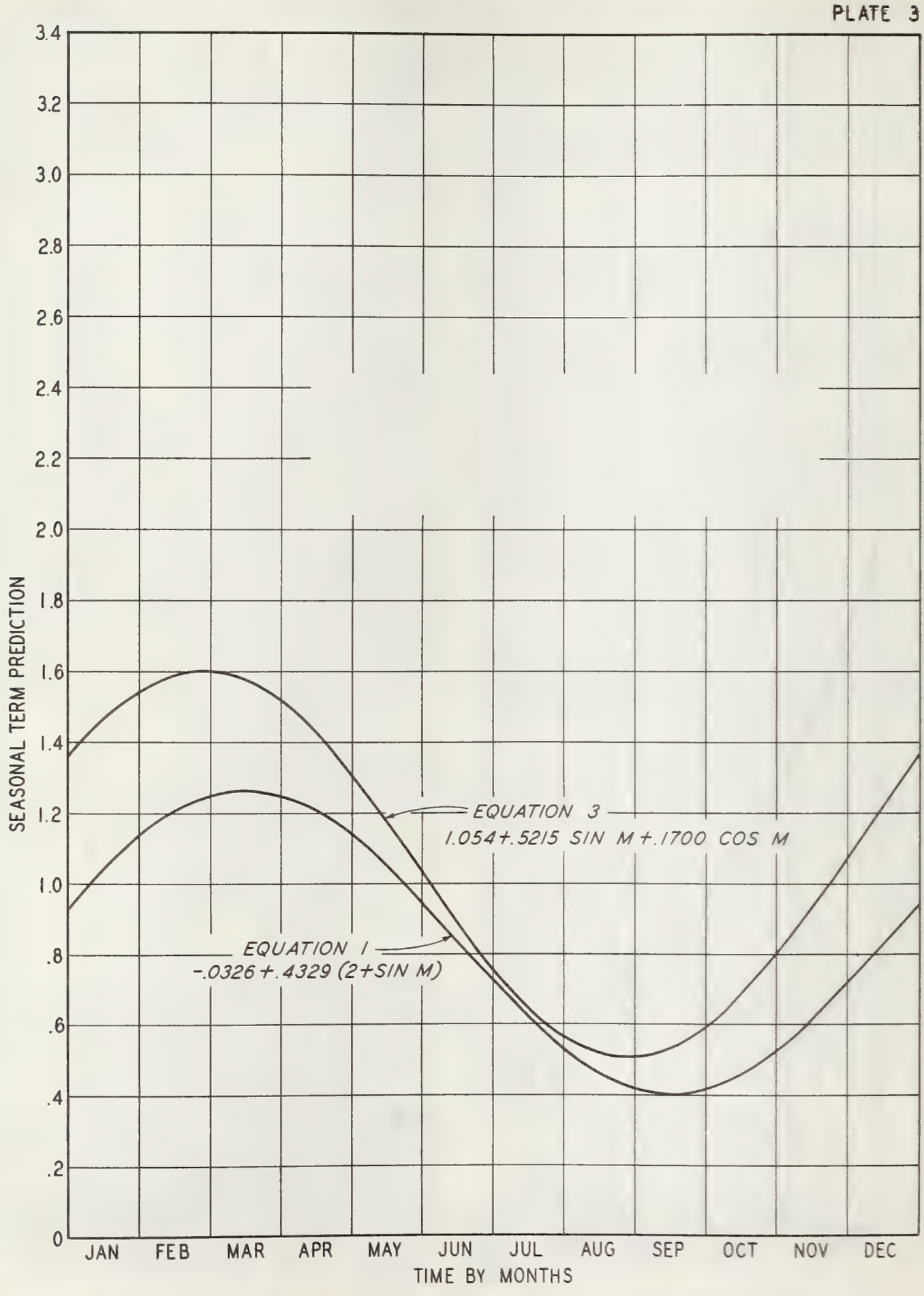

FIGURE 3. — Chestuee Creek Watershed: Comparison of season-regression terms (equation 1 vs. equation 3).

PLATE 4

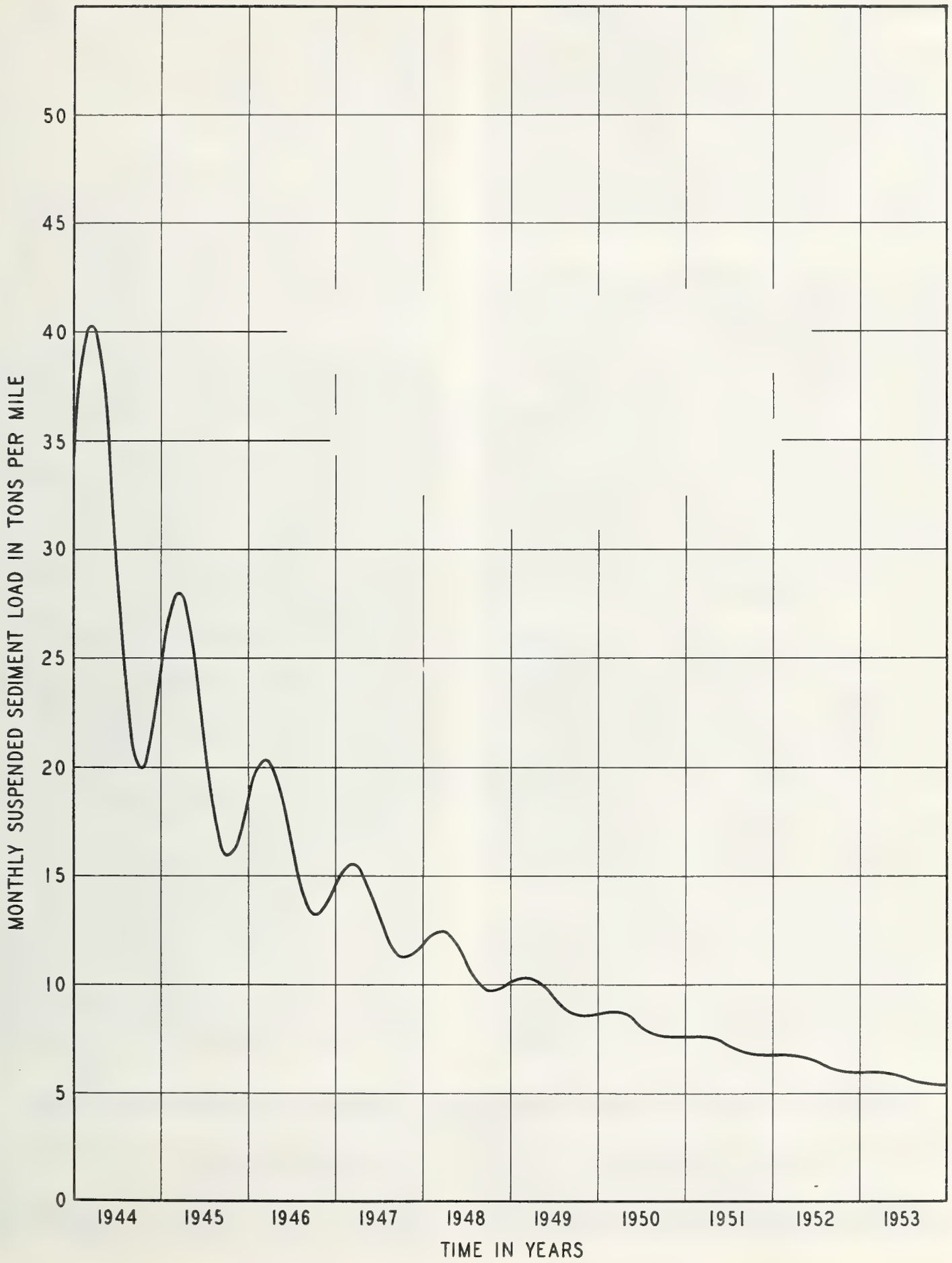

FIGURE 4. — Chestuee Creek Watershed: Suspended sediment prediction by time regression function modified by season (equation 4).

$$Y = -12.19 - 0.0956\, P_1^2\, P_2^{1/3} + 0.4613\, P_1^2\, P_2^{1/3}\,(2 + \sin M) + 14.55e^{-0.8319T} + 9.853e^{-4.581T}\,(2 + \sin M) \quad (4)$$

The coefficients of the rainfall terms are not much different from those in equation 1. However, the arrangement of coefficients in the two time-regression functions indicates that this relationship has undergone considerable change. Figure 4 is a plotting of the composite time-regression function of equation 4. It is evident that there is a considerable seasonal difference in the amount that the sediment load was reduced.

To estimate the magnitude of reduction in sediment loads by seasons, equation 4 can be used as a prediction device. If appropriate mean monthly rainfall values are used for the precipitation variables and time is allowed to vary from a specific month in 1944 to the same month in 1953, estimates can be made of the magnitude of the change in sediment transport by seasons. The equation indicates that average rainfall would have resulted in a total of 106 tons of sediment per square mile in March 1944 as compared to 72 tons in March 1953 for a reduction of 34 tons, or 32 percent. Similarly, for September the equation indicates that average rainfall would result in a total load of 16.9 tons per square mile in 1944, but only 2.4 tons in 1953 for a reduction of 14.5 tons, or 86 percent.

Thus, a slight modification in the equation, made because the original was not sufficiently versatile to describe desirable relationships, has resulted in a far better understanding of the sediment transport characteristics of the watershed. The monthly sediment reduction indicated by the original equation was 29 tons per month on the average, whereas that indicated by the modified equation varied from a high of 34 tons in March to a low of about 14.5 tons in September. This latter result not only gives more insight to the changing sediment transport relations; it is also more realistic.

Although statistical interpretation of the results obtained from fitting the four equations to the Chestuee Creek data is beyond the scope of this paper, certain interesting implications can be drawn. In all three of the equations fitted by the computer, very little improvement over the fit of the original equation was accomplished. The multiple-correlation coefficient for all equations was 0.9 and the standard error about 25 tons per square mile. The fact that significant improvements in the fit were not realized, despite the fact that the number of restraints on the equation were varied and improvements made in the model, indicates that this is about the limit of the adjustment of these data that can be made with these variables. If we consider that the variables used in the equation are monthly totals, although sediment transport is primarily a function of individual storm characteristics, a substantial standard error is not unexpected. Further improvements in the fit of the model could probably be made only if additional information were introduced.

Conclusion

The electronic computer has provided the hydrologist with the analytic means of solving equations of an extremely complex nature. No longer need he be limited by linear equations or transforms to linear equations. Mathematical models can be developed that utilize to a much greater degree the complex functional relationships that exist in nature. On the other hand, the efficiency and speed of the computers allow a direct approach to empirical model building. The data can be used to aid the hydrologist in building a model. The results of one fitting of an equation to the data can be used as a guide to improving the basic model. Model building in this manner can lead the hydrologist to a much better understanding of the relationships involved. It enables the hydrologist to construct his model in such a manner that the final equation, even though entirely empirical, can yield physically interpretable results, because the model can be based on logical and rational considerations. The development need not be biased by a foreknowledge of requirements of simplicity and linearity.

An example of model manipulation, utilizing monthly suspended sediment data, is shown in the paper. The use of a month as a time base for suspended sediment measurements not only precludes any attempt to find physical causal relationships; it severely limits the number of meaningful parameters available. Despite these limitations, results are obtained from these data that lead to a good understanding of the sediment transport characteristics of a watershed.

Land Use and Ecological Factors in Relation to Sediment Yields

[Paper No. 11]

By Otis L. Copeland, Jr., *chief, Division of Watershed Management Research, Intermountain Forest and Range Experiment Station, Forest Service, U.S. Department of Agriculture*

America's development for more than three centuries has been advanced largely by the prodigious utilization of her natural resources, especially soil, water, timber, forage, oil, and

minerals. But this progress created attendant problems of depletion or deterioration of these basic resources. Important resource problems that concern this audience are the impoverishment of productivity, impairment of hydrologic processes, aggravation of flood peaks, and the increasing menace of sedimentation as a result of land use.

This discussion deals with ecological factors and land uses that influence sediment yields, and although the terms "soil erosion" and "sediment yield" are not synonymous, they are closely related and may occasionally be used interchangeably. "Sediment yield," as used in this paper, is the quantity of transported soil material resulting from land use and channeling of valley fill, the value of which varies with the size and condition of any given drainage area under consideration. "Soil erosion" refers to the detachment and movement of soil particles on-site, but does not necessarily imply movement into channels. Quantitatively, the amount of erosion may greatly exceed sediment yields measured in channels or at off-site locations.

Erosion and sediment production are not strangers to agricultural scientists and land managers, and their relation to land use is nothing new in this country and elsewhere. To discuss them is like "an old refrain" played again. The antiquity of the concept of land use and soil erosion and thus sediment yield is aptly illustrated by the Union of South Africa's appointment in 1920 of a commission to inquire into ways of improving water resources of the farms. Among the commission's findings were these: "Soil erosion is caused by reduction of the Country's mantle of vegetation." And further, "The soil belongs to the nation, not to the individual, and its dissipation through erosion is a national calamity that demands the aid of everyone to combat it" (*5*).

Recognition of sediment yield problems and the need for corrective action has progressed steadily in this country. Congressional action creating the national forests, establishing the Soil Conservation Service and later the organization of soil conservation districts, passing the Taylor Grazing Act, the C.C.C. program, and the various flood control and watershed protection programs are ample evidence of public concern and a desire to provide for remedial action. And, while no one would question the direction toward which the programs were oriented, there is serious concern about the slow rate of progress being made in applying knowledge now available to assure a coordinated and integrated approach to problems of land use to control sediment production at its point of origin. Phenomenal population pressures give an added sense of urgency to the need for accelerated progress in remedial programs.

Sediment production as a result of erosion on damaged watersheds indicates the loss of control over water in its contact with the soil. It results in such visible kinds of damage as silted reservoirs, clogged transmission structures, damaged hydroelectric facilities, buried valley lands, destroyed fish habitat, and scoured channels. Accompanying these visible damages, but far more insidious, is the gradual loss of the soil resource and consequent lowering of productivity.

In this paper, such land uses as logging and road construction, grazing, and recreation are considered. Ecological factors to be considered include vegetation, geological parent material and soils, slope, precipitation, and fire. Man may logically be regarded as an ecological factor because he may profoundly influence the environment through his various uses of the land. Ample evidence supports the contention that man has been a potent force in accelerating sediment yields because of his activities. Too often he has failed to reckon with the powerful forces of nature. This is not to say that man has extended his horizons of land use wrongfully — rather, let us say, too often unwisely. The crux of the situation is that public pressures will not long tolerate the continued failure to apply the protection necessary to maintain stability on our watersheds and thereby keep sediment production in check.

To achieve rapport in this discussion, a differentiation should be made between normal and accelerated sediment yields, because it is the latter with which we are concerned. This differentiation has been stated succinctly by Bailey (*2*):

> Every watershed is the product of many natural processes, including rock weathering, soil formation, erosion, and biotic succession, all of which have been operating under the impact of climate over the ages. Because of local differences in the climate, the resistance of rock to weathering, and such other features as the aspect, and length and steepness of slopes, differences have developed in the character of the plant cover and soil mantle, and in runoff and erosion. In some drainage basins, streams fluctuate but little either seasonally or annually, and carry negligible quantities of sediment. Others are frequently in violent flood stage and are generally muddy. Still others exhibit runoff and siltation characteristics between these extremes. Where such variations are but manifestations of different degrees of control established by nature, there is little that watershed management can do toward their control.
>
> It has been definitely established, however, that many of the floods and much of the sediment load carried by streams are not of normal proportions, but have been magnified by impairment of the plant cover and soil mantle on the watershed slopes and in valley bottoms. Watershed management can reduce such discharges and siltation rates, but only to the extent that they have been increased by watershed deterioration above the normal.

Notable progress has been made in curbing and controlling sediment yields through land management augmented by engineering structures. Watershed management and rehabilitation practices, now being applied more widely than ever before, are designed to control sediment at its source. These practices are being improved as research continues to reveal more definitely the interrelations among climatic, edaphic, and vegetation factors that control the swing of the pendulum of soil stability. Let us now examine the influence of ecological factors and land uses on sediment yields.

Precipitation

The magnitude of kinetic energy generated in 30 minutes by different rainfall intensities is shown in figure 1, constructed from basic energy data by Wischmeier and Smith (*19*). This graph shows that a rain falling at an intensity of 0.1 inch per hour produces in 30 minutes kinetic energy equaling 58,500 foot-pounds. A rain with an intensity of 2 inches per hour generates, in 30 minutes, kinetic energy of slightly more than 2 million foot-pounds. This would be sufficient to raise an acre furrow-slice 1 foot high. A rain of 8 inches per hour intensity would produce, in 30 minutes, enough energy to raise an acre-foot of soil almost 3 feet. While it is obvious that this is cumulative energy generated over the stated period, these values emphasize the tremendous force unleashed by torrential rainfall. It is dissipation of this energy toward which land use must be oriented.

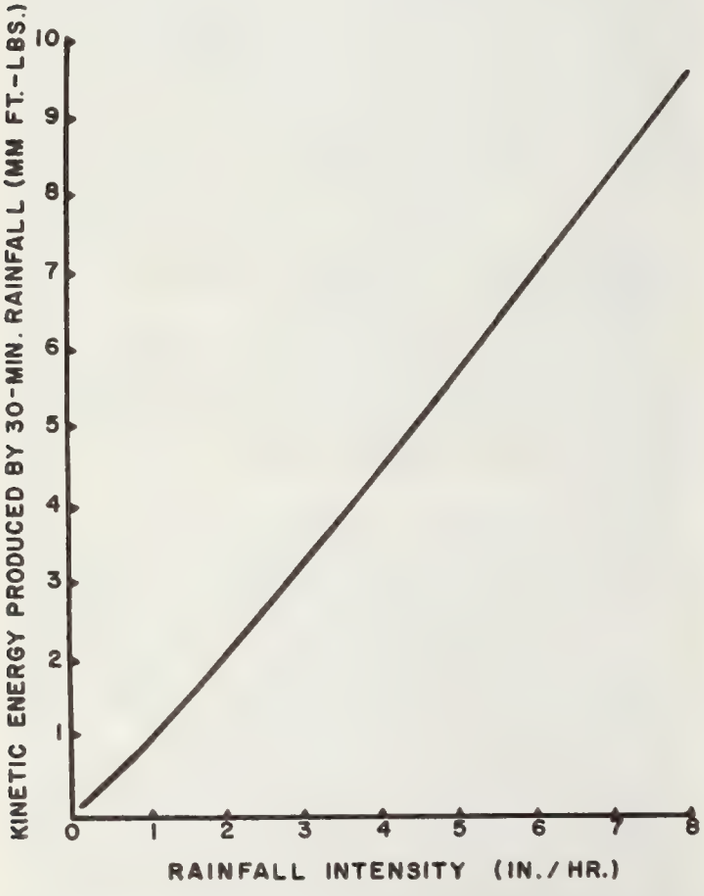

FIGURE 1. — Kinetic energy produced by 30-minute rainfalls of different intensities.

Although the last rainfall intensity mentioned above may be considered extreme, high-intensity storms are common over much of the United States and particularly in the Intermountain States. These storms, usually of short duration and limited extent, produce rain that is a powerful eroding agent. Examples of high-intensity storms are shown in figure 2; here it may be observed that the maximum intensity occurred at different time periods in each storm. Seldom do these storms occur when the soil mantle is saturated; rather they occur when the mantle has the capacity to store several inches of water. The critical characteristic is that the rainfall intensities of these cloudbursts frequently exceed the infiltration rate of the soil, especially when plant cover is depleted and overland flow is then inevitable.

Soils

A consideration of soils in relation to sediment yield introduces an almost infinite number of conditions that affect the infiltration of water very much and, conversely, the amount of overland flow and sediment yielded. Among these are texture of surface soil, inherent structure and consistence, depth of soil (especially to restrictive layers), porosity, and the percolation rate of subsurface horizons. Collectively, these factors determine the receptivity of water into that topmost thin skin of the soil mantle.

Soils derived from different geologic parent materials often exhibit varied capacities to infiltrate precipitation. These differences may be inherent or induced, and are illustrated by the data below obtained on the Boise River watershed (*15*).

Infiltration rate (inches per hour):	*Granite*	*Basalt*	*Sedimentary rocks*
Soil in normal condition..	6.9	7.1	5.6
Soil in eroded condition..	2.8	1.5	2.0
Watershed in each type (percent)	75.4	9.4	15.2

Erosion by water of the more receptive surface horizon has reduced infiltration 60, 80, and 64 percent, respectively, on soils derived from granite, basalt, and sedimentary rocks. Further reductions are possible as erosion exposes less permeable horizons and as the total infiltration capacity is diminished by creating shallower profiles. The greatest reductions in infiltration rates after erosion occurred in the finer tex-

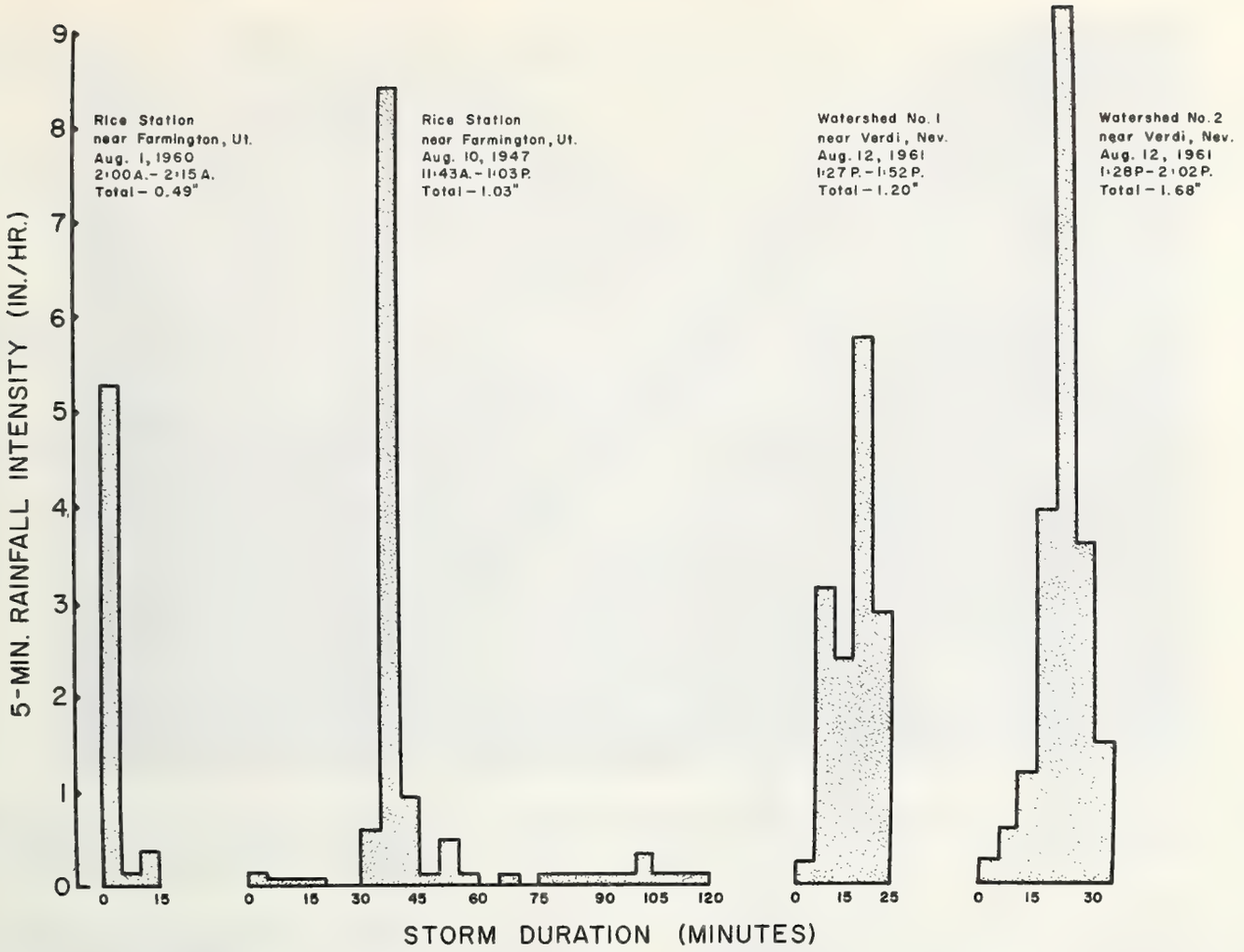

FIGURE 2. — Rainfall intensities of three storms (one measured at two locations) by 5-minute periods.

tured soils derived from basalt and sedimentary rocks.

Packer [1] investigated six major soil groups in the northern Rocky Mountain region to determine the levels of resistance to erosive cutting by overland flow on logging roads. He found that the order of increasing erodibility of the soils was hard sediments, basalt, granite, glacial silt, andesite, and loess. The general prevailing characteristic of the more erodible soils is their tendency toward single-grained structure and uniform texture.

Sediment yields are also influenced by other soil characteristics. Swelling of soils upon being wetted, compaction by trampling of animals, by machinery, or the impact of falling rain increase sediment production by reducing infiltration.

As soil bulk density increases, there is a corresponding decrease in porosity, a reduction in infiltration, and an increase of overland flow. Where ground cover is depleted to less than the minimum density required to protect the soil, sediment yields from relatively small areas may increase fantastically. Curves in figure 3,*A* show how sediment yields vary in relation to bulk density. These curves are based upon results from 160 plot tests of infiltration carried out by Meeuwig [2] in which 2.5 inches of simulated rainfall were applied in 30 minutes. Under low densities of ground cover, typical of deteriorated sites, there may be as much as a fourfold increase in sediment yields as soil bulk density increases from 0.8 to 1.4. The inset correction factor curve shows that sediment production increases sharply on sites with depleted cover as slope gradients exceed about 25 percent. Where ground cover is complete, changes in sediment yields with increasing slope gradients are minor. This point is discussed more fully later in this paper.

On soils derived from sedimentary material in Montana, similar pronounced increases in eroded sediment in relation to bulk density are shown in figure 3,*D*. Although the amounts are

[1] Personal communication, P. E. Packer, September 1962.

[2] Personal communication, R. O. Meeuwig, September 1962.

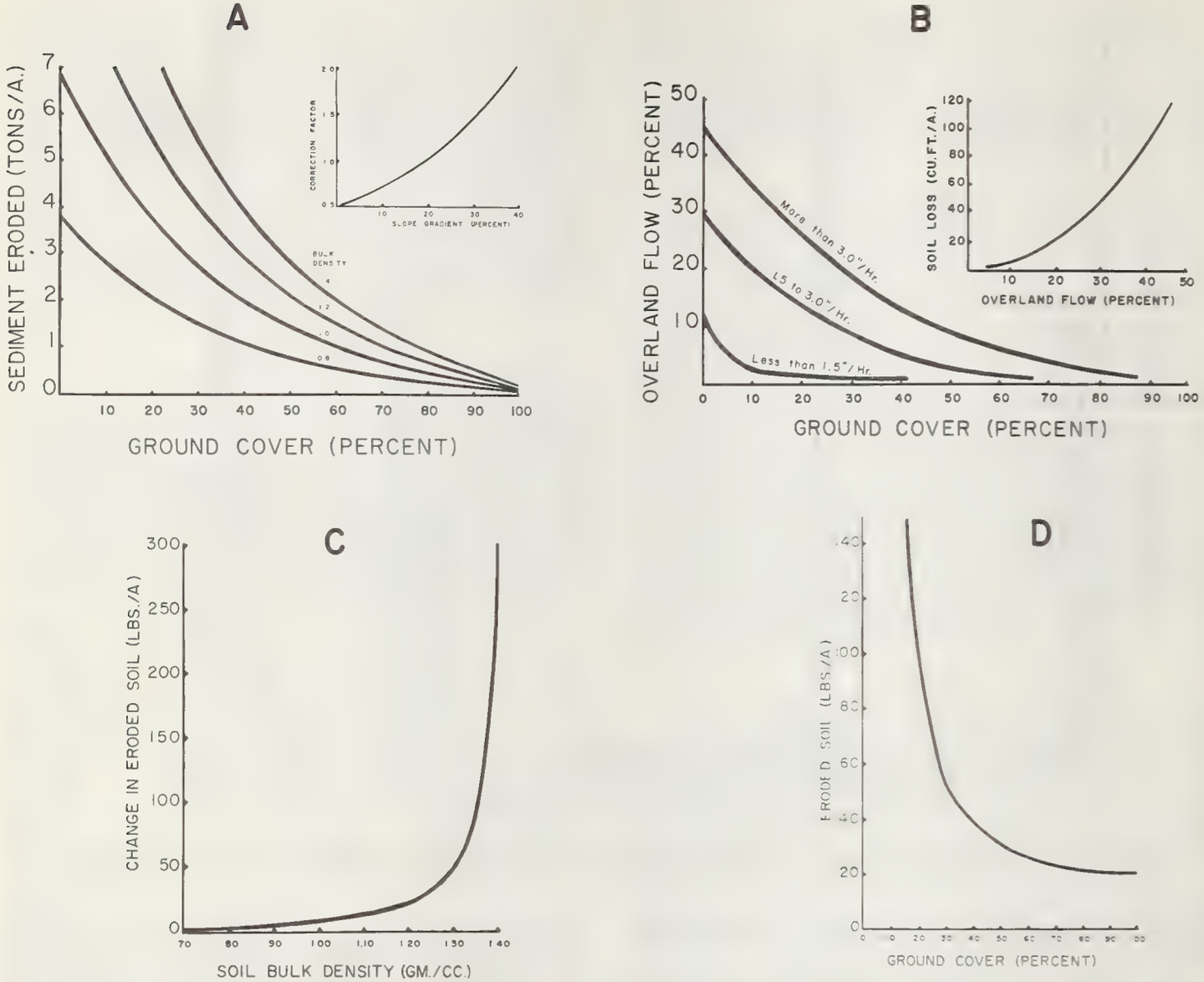

FIGURE 3. — *A*, Sediment eroded from central Utah rangelands in relation to ground cover and soil bulk density. The inset curve is a correction factor for sediment yield as affected by slope gradient. *B*, Overland flow by different storm intensity classes as a function of ground cover. The inset curve relates sediment to overland flow (Davis County Experimental Watershed, Utah). *C*, Changes in the amount of sediment produced in relation to soil bulk density on elk winter range in Montana. *D*, Sediment production as a function of ground cover at a mean bulk density of 1.14 on elk winter range in Montana.

considerably less than those in figure 3,*A*, the trend is the same (*12*). Packer concluded that one prerequisite to maintaining soil stability on this elk winter rangeland is a soil bulk density of 1.10 or less.

Vegetation

Residual soils result from the weathering of parent material as influenced by time, climate, relief, and vegetation. The retention of the weathered material in place and its maturation as soil in the face of vagaries of weather and other natural phenomena depend upon simultaneous vegetation development, especially on steep mountainous areas.

Bailey (*1*) measured numerous slopes in Idaho and Utah to determine the maximum angles of repose of those vegetated and barren. He found that barren talus-covered slopes stood at gradients between 68 and 80 percent, or at an angle of about 36°. In contrast, densely vegetated slopes underlain by fine-textured soils derived from material similar to the talus did occur on slopes with gradients up to 173 percent, or 60° (fig. 4). Without the stability afforded by vegetation, these soils could not have accumulated. They would have been displaced by gravity or would have been blown or washed away, and the resulting slope gradient would not have exceeded that of the talus.

FIGURE 4. — Vegetated slopes underlain by fine-textured soils with a well-developed profile are stabilized at angles far steeper than the angle of repose for similar nonvegetated material. These slopes in south-central Idaho have gradients that exceed 100 percent.

Despite many variations in types of vegetation, its role in promoting soil stability is multifarious. It binds the soil, promotes infiltration, dissipates raindrop energy, develops greater porosity, and enriches the surface soil. Thus, the maintenance of an adequate cover of vegetation is the key to stability of soils on sloping lands. The presence or absence of vegetation determines whether the soil stability balance is maintained or whether degradation sets in.

Watershed rehabilitation treatments were developed and applied to damaged headwater areas of numerous drainages in Davis County, Utah, beginning in 1933. These damaged areas comprised less than 10 percent of the total area but were sources of unprecedented mud-rock floods. Plots were established in 1936 on the cover-depleted and eroding flood-source areas and on nearby well-covered noneroding lands that were considered to be nonflood-source areas. Overland flow and sediment yield data in table 1 gathered from these plots substantiate the role that vegetation plays in protecting the soil.

TABLE 1.—*Summer storm rainfall and resultant overland flow and soil losses from Parrish plots, Davis County Experimental Watershed, Utah*

Storm dates	Total rainfall	Non-flood-source area		Flood-source area		Artificially denuded	
		Overland flow	Soil eroded	Overland flow	Soil eroded	Overland flow	Soil eroded
	In.	*Pct.*	*Cu.ft./acre*	*Pct.*	*Cu.ft./acre*	*Pct.*	*Cu.ft./acre*
July 10, 1936	1.14	0.7	0	42.8	181.5	.....	.....
July 16, 1936	.89	.4	0	43.4	153.6	.....	.....
July 28, 1936	1.21	.2	0	33.0	83.2	.....	.....
Aug. 18–20, 1945	3.09	.5	0	24.3	92.8	.....	.....
July 10, 1950	.70	.9	0	12.6	(1)	61.3	215.3
Aug. 19, 1951	1.15	.6	0	8.4	(1)	46.6	186.2
Aug. 4, 1954	1.17	.4	0	3.8	(1)	31.3	91.3
Aug. 19–20, 1959	.98	.6	0	2.3	(1)	43.7	98.4
Sept. 3, 1960	.63	.....	.....	.....	.....	31.0	110.0
Aug. 25, 1961	.64	.....	.....	.....	.....	28.6	89.2
July 13–14, 1962	2.59	.....	.....	.....	.....	39.0	401.3

[1] Trace.

Nonflood-source plots on the Davis County Experimental Watershed supported vegetation of a density capable of limiting overland flow to an average of about 0.5 percent of 10.33 inches of summer rainfall from eight major erosion-producing storms from 1936 to 1959, most of which fell at high intensity. For 25 years, no measurable sediment has been yielded from any of these plots. Nearby flood-source plots yielded 511 cubic feet of sediment per acre during the first four storms; 36 percent of the total rainfall was lost as overland flow. Improvement in plant cover on these former flood-source plots during the last 15 years has reduced overland flow to 6.8 percent, and sediment yields have been nil. Contrasted to this situation is that of other plots denuded artificially. Overland flow from these plots averaged 40.2 percent of the rainfall from seven major storms from 1950 to 1962, with an average sediment yield of 170 cubic feet per acre per storm.

Relation of Vegetation to Overland Flow and Erosion

There is little question of the beneficial effect of ground cover in controlling erosion. Numerous studies have demonstrated repeatedly that major reductions in vegetation and the frequent accompanying compaction of soils impair desirable hydrologic processes, especially those involving safe disposal of precipitation. Despite knowledge of these consequences, only limited research has been directed towards establishing watershed protection requirements for the numerous soil and vegetation complexes that characterize forest and range lands. Much of this effort has been centered in the Intermountain States.

Studies on five major range vegetation-soil complexes to determine requirements for controlling overland flow are summarized briefly below.

Wheatgrass-Cheatgrass Range

Studies have shown that, for effective control of overland flow and erosion on coarse-textured, single-grain-structured soils derived from granite, no less than 70-percent ground cover is required, and that bare openings between plants or patches of litter should not exceed 4 inches on wheatgrass sites and 2 inches on cheatgrass sites (*11*). These conditions have provided effective control of overland flow under a 30-minute simulated rainfall of 3.7 inches per hour. Observations on extensive similar sites subjected to natural high-intensity rainfall substantiate the validity of these minimum protection requirements.

Because of a contention that high-intensity rainfall bursts seldom exceed 5 to 10 minutes, further study was made to determine whether less stringent ground cover requirements would suffice. Figure 5 relates cumulative overland flow and cumulative sediment yields to ground cover by 5-minute periods (*17*). These curves show unmistakably that, irrespective of which 5-minute period is considered, both overland flow and sediment increased only slightly as

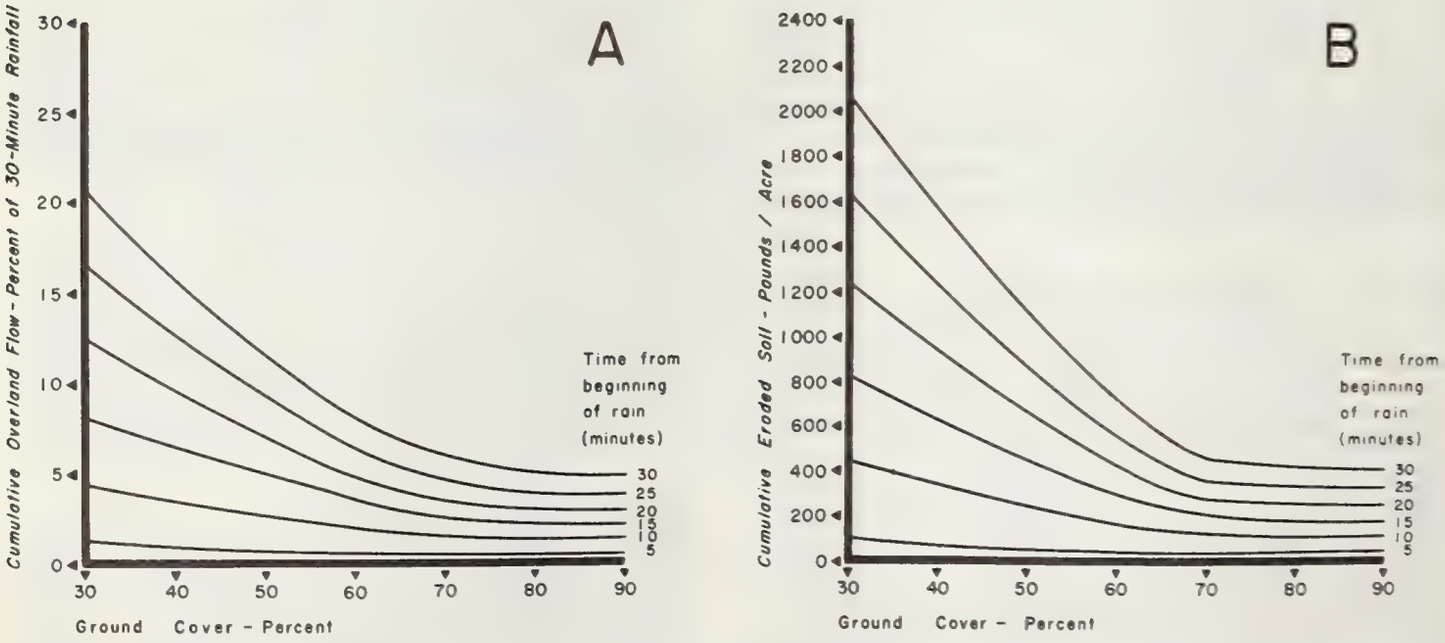

FIGURE 5. — *A*, Cumulative overland flow; and *B*, cumulative soil eroded by a simulated rainfall of 1.87 inches for 30 minutes by 5-minute periods in relation to ground-cover density. This study was conducted on wheatgrass range sites on soils derived from granite (*17*).

ground cover was reduced from 90 to 70 percent; but both increased tremendously with further reductions in ground cover below 70 percent.

Subalpine-Herbaceous Range

Subalpine-herbaceous range in Utah is commonly underlain by fine-textured clayey soils derived from limestone and carbonaceous shales. Since 1915, studies of the effects of ground cover changes, through land use, on sediment yields have been underway on the A and B watersheds (*10*). Overland flow and sediment production from these watersheds is summarized in table 2 by time periods.

Upon establishment, these two watersheds were protected from grazing; later, it was imposed as a treatment on watershed B. To facilitate the comparison of the effects of treatments, the history of these two watersheds was divided into six periods. In the first period, watershed A, in an extremely depleted condition (16-percent plant cover) produced 6 times more runoff and more than 5 times as much sediment as did watershed B (40-percent plant cover).

TABLE 2.—*Average plant cover density and average annual summer storm runoff and sediment yield on watersheds A and B by periods, 1915-58 (10)*

Period	Years	Average plant cover		Average annual summer storm runoff			Average annual summer storm sediment		
		A	B	A	B	Ratio A:B	A	B	Ratio A:B
		Pct.	*Pct.*	*Cu.ft./acre*	*Cu.ft./acre*		*Cu.ft./acre*	*Cu.ft./acre*	
1	1915–20	16	40	913	153	6.0:1	134	25	5.4:1
2	1921–23	30	40	922	260	3.5:1	105	37	2.8:1
3	1924–30	40	40	362	171	2.1:1	24	10	2.4:1
4	1931–45	40	30	445	556	1.0:1.2	21	29	1.0:1.4
5	1946–52	40	16	64	288	1.0:4.5	3	36	1.0:12.0
6	1953–58	40	50	292	92	3.2:1	10	1	10.0:1

During the second and third periods, vegetation on watershed A improved, resulting in less runoff and sediment. In the fourth period, watershed B was grazed lightly to reduce the plant cover to 30 percent. Immediately, the magnitude of overland flow and sediment production reversed, and B produced more of each than did A. In the fifth period, B was overgrazed to render it comparable to the condition of A in the first period (16-percent plant cover). Overland flow and sediment yields increased markedly. Sediment yield on B surpassed the yield from A at the beginning of measurements. After this period, B was contour-furrowed and reseeded. Again, the sixth period brought a profound reversal of overland flow and sediment yields — an improvement that far exceeds the original condition.

This oldest continuous pair of range watersheds in the United States has demonstrated remarkably well that ground cover and land use are inextricably linked to proper functioning of hydrologic processes.

Aspen-Herbaceous Range

The aspen-herbaceous range has been the locus of severe mud-rock floods in northern Utah. On the Davis County Experimental Watershed, where the fairly deep, intermediate-textured soils have been derived chiefly from gneisses and schists, records obtained continuously since 1936 from 16 plots ranging from $\frac{1}{40}$- to $\frac{1}{10}$- acre in size and supporting varying amounts of ground cover, provide information upon which watershed protection requirements can be based (*9*). When only those summer storms that produced sediment from the plots are considered, it is evident that by limiting overland flow to no more than 5 percent of rainfall depth, a reasonable degree of stability can be maintained on this type of range. These long-term records provided a basis to develop ground cover requirements to provide protection under different intensities of rainfall. Relations of overland flow to ground cover by storm intensities are shown in figure 3,*B*. For 5-minute intensities of less than 1.5 inches per hour, a ground cover density of at least 5 percent was required to restrict overland flow to 5 percent or less. Five-minute intensities of 1.5 to 3.0 inches per hour required 45-percent ground cover, and those in excess of 3 inches per hour required at least 65-percent ground cover to achieve desired minimum overland flow.

Grass-Forb Winter Range

Study sites were located in Montana on intermediate-textured soils derived from mixed sandstone and shale. These grass-forb vegetated slopes are the winter range for large numbers of elk. Over a 2-year period, plots with a ground cover of 70 percent yielded almost as little sediment as those with 100 percent (*12*). But as ground cover was reduced below 70 percent, sediment eroded from plots increased markedly (fig. 3,*D*).

Results from these four widely separated areas representing diverse vegetation, soil, ele-

vation, and precipitation characteristics, suggest strongly that a minimum ground cover density is required to control overland flow and erosion from these ranges. They also emphasize the relation of bulk density and slope gradient to sediment production under conditions of depleted plant cover. Where complete cover exists, these two factors are of much less significance.

Effects of Land Uses on Sediment Yields

Logging

Albeit numerous studies have determined the amount of soil disturbance created by logging and fewer studies have sought to measure actual sediment yields, we still lack thoroughly comprehensive programs of research to determine quantitatively the integrated effects of various methods of logging upon watersheds. This situation may aptly be attributed to the almost overwhelming complexity of the problem, and perhaps, for this reason, the effects of logging have been studied piecemeal, or by facets.

Soil disturbance in itself is not inimical to watershed stability. It becomes so when erosion produces sediment that is transported by runoff to lower lying channels. Some disturbance bares the soil only temporarily and is silviculturally desirable for the natural regeneration of forest species. Often, however, extreme disturbance also results in destruction of undue amounts of young or lesser vegetation, thus prolonging the time of natural recovery or increasing the effort required to stabilize the disturbed areas artificially. Frequently, on-site disturbance is a forerunner of sedimentation. This then leads to off-site disturbance, and a great need exists for recognizing the potential for this situation to develop. When it does develop, then there is synonymy between soil disturbance and sediment yields.

The magnitude of soil disturbance, its potential seriousness, and the conditions under which it occurs warrant brief review. Results of a study in Oregon and Washington by Garrison and Rummell (*6*) showed that 15 percent of logged areas suffered deep disturbance created by tractor logging, whereas cable logging produced equal disturbance on only 1.9 percent of the area. Deep soil disturbance on slopes exceeding 40 percent was 2.8 times greater than on gentler slopes.

Another study in the Pacific Northwest (*14*) revealed that clear cutting 12 units on the H. J. Andrews Experimental Forest disturbed 8.8 percent of the total area by road building and another 3.6 percent by the construction of landings, for a total of 12.4 percent.

On the Fernow Experimental Forest in West Virginia, average turbidity of streamflow during logging operations was 490 p.p.m.; 1 year after logging, it was 38 p.p.m.; and 2 years after logging, 1 p.p.m. On one drainage where little planning of the logging was done, maximum turbidity reached 56,000 p.p.m. On a comparable drainage where a well-planned logging system was applied, maximum turbidity was only 25 p.p.m. (*18*).

Haupt (*7*) reported that the area of soil bared by harvesting ponderosa pine on 16 compartments, ranging from 30 to 78 acres, on the Boise Basin Experimental Forest, in south-central Idaho, was closely related to the number of trees and volume of timber removed. Stem selection was used in eight compartments; group selection in the others. Under the two harvesting systems, an increase from 1,500 to 6,500 trees cut per square mile resulted in increased soil baring from 29 to 114 acres for single tree selection, and from 29 to 84 acres for group selection. On these moderate slopes of fairly deep soil, single tree selection and harvest produced the greater soil disturbance. Total area of soil disturbance for all compartments under both systems averaged 8.1 percent.

Facilities for making postlogging sediment yields have been in operation in all compartments since 1954. Because of the carefully designed logging plan and generally good application on the ground, measurable quantities of sediment have been recorded in only two compartments. In both, this sediment resulted from the encroachment of a logging road on a channel.

In marked contrast to some of the above examples, on the Stamp Creek timber sale covering 800 acres on the Chattahoochee National Forest in Georgia, logging was required in every month of the year under a carefully developed plan. Logging resulted in an average sediment content of the streamflow of 5 as compared with a sediment content of 4 p.p.m. on a stream from an unlogged watershed (*4*). This sediment content is only one-half that permitted by the standards for drinking water published by the U.S. Public Health Service.

Road Construction

Of man's activities on forested lands, disturbance caused by road construction is the greatest precursor of sediment yields. A few examples aptly illustrate the seriousness of this problem, which is most acute during and for the first few years after road construction.

On the H. J. Andrews Experimental Forest (*16*), 1.7 miles of logging roads were constructed in a 250-acre watershed, exposing mineral soil over 6.2 percent of the area. For 6 years preceding road construction, sediment content of the streamflow never exceeded 200 p.p.m. even after high-intensity storms. Rains commenced shortly after road construction, and suspended sediment of the streamflow exceeded

1,700 p.p.m. An adjacent control watershed without roads produced a sediment content of only 22 p.p.m. after the same rainstorm. Thus, the first storm on this watershed after road construction produced a streamflow sediment content 81 times greater than the undisturbed watershed. About 2½ years later, the logged watershed produced sediment at the rate of 4.7 times as much.

One other example of the deleterious effects of roads on sediment production is provided in south-central Idaho by experimental drainages situated on highly erodible coarse-textured soils derived from granite.[3] Although no sediment was yielded from areas without roads, three adjacent watersheds, in which jammer logging roads were constructed without any extra precautionary stabilizing or protective treatments, produced yields of 12,400, 8,900, and 89 tons per square mile in one season. These sediment yields are attributed to the lack of adequate cross drains coupled with uncommonly heavy precipitation.

Fire

When an area is subjected to wildfire or prescribed burning, vegetation is lost. But because prescribed burning is designed as a controlled operation, attention in this discussion is centered on wildfires, over which no planned control is exerted.

Sartz (*13*) measured soil losses that occurred after wildfire in the Douglas-fir region of the Northwest. Erosion of burned-over transects caused 0.07- to 0.18-foot reductions in the surface soil levels. These reductions varied directly with changes in slope gradient.

The cutting and burning of slash of young pine in Colorado increased sediment production 4½ times or from 640 pounds to 3,000 pounds per acre over the average for a 10-year period (*8*).

In the summer of 1959, 9,517 acres of foothill rangeland situated above the city of Boise, Idaho, was burned by a wildfire. Little vegetative cover was left. A few weeks after the burn, an extremely high-intensity rainstorm struck the area. Within hours, lower lying farmland and residential areas were inundated by sediment, bouldery debris, and floodwaters (fig. 6). The city of Boise suffered flood damages estimated at one-half million dollars. Storm-cut rills and gullies, ranging from ½ inch to 8 inches deep and from 1 inch to 18 inches in width, averaged 83 per 100 feet along the contour on some of the denuded watersheds. These burned watersheds produced two other floods during the next month that added to the initial flood damages.

[3] Personal communication, H. F. Haupt, September 1962.

FIGURE 6. — Pastures and farmland near Boise, Idaho, covered 1 foot to 3 feet deep with debris from the August 20, 1959, flood.

In August 1961, a sediment-laden flood originated on the burned slopes of two experimental watersheds in Dog Valley, near Verdi, Nev. The year before, an intensely hot wildfire swept over all of one and a part of the other watershed, obliterating practically all of the ground cover and killing most of the overstory timber (fig. 7). The maximum 5-minute intensity of the storm was 9.24 inches per hour (fig. 2).

Sediment filled the stilling basins back of two weirs, and spilled into channels below for a distance of 1 mile or more. Watershed No. 1, completely burned, produced about 16,100 cubic feet of sediment from its 242 acres for a rate of 0.97 acre-foot per square mile. Watershed No. 2 (300 acres), of which only a portion was burned, produced 10,125 cubic feet or 0.49 acre-foot per square mile. The unburned portion produced scarcely a trace of sediment. The sediment production rates for a single rainstorm of 30 minutes' duration reflect the enormous erosion potential unleashed by the destruction of vegetation.

Excessive Grazing

Unregulated and excessive grazing proves destructive and has been shown unmistakably to be the cause of sediment yields far in excess of the normal. This is especially true for steep rangelands of the Western States. A marked increase in flooding, channel cutting, and sediment production after the introduction of herds onto these lands is a matter of historical record. Excessive grazing by livestock herds in the Santa Clara, Kanab, Long Valley, Paria, Escalante, Kitchen Valley, Ford, Parrish, Farmington, Pleasant Creek, and countless other drainage basins in Utah alone, induced rapid deterioration of vegetation. Floods, heavily laden with sediment, issued from these overgrazed

FIGURE 7. — A densely vegetated forest cover obliterated by fire. The stable slopes and channels were transformed by a high-intensity storm into raw, eroding areas that continue to yield large quantities of sediment.

lands within varying periods of time — some as short as 7 years (*3*).

In many areas this severe use, coupled with relatively short growing seasons often punctuated by droughts, soon resulted in diminution or destruction of plant cover. Under these conditions, "the rains came," and localized storms of short duration produced rainfall intensities frequently in excess of the infiltration capacities of the soil. Overland flow developed, erosion followed, and the concentration of water and sediment resulted in flood discharges of great violence.

One example will suffice to illustrate the disastrous effects that may ensue. In the Parrish watershed in Davis County, Utah, overgrazing and some man-set fires depleted drastically the natural vegetation over the 1,378-acre drainage basin, especially in the headwaters area. Virtually all vegetation was destroyed on areas aggregating only 12 percent of the watershed (fig. 8). Four mud-rock floods issued from this watershed in 1930, causing more than one-third of a million dollars' damage to property and developments below the mouth of the canyon. In that one summer, the short-time sediment yield derived from both damaged slopes and channel fill, when calculated on a square-mile basis, equaled 153 acre-feet. In sharp contrast, the nearby undamaged Morris Creek watershed has produced sediment at the rate of only 0.0025 acre-foot per square mile per year. The literature abounds with references to other floods after overgrazing.

Recreation

Along with old, extensive, and well-established land uses, we are now confronted with another rapidly expanding land use that bodes a serious erosion and sediment yield problem. Although not as widespread geographically as other uses, the sediment production problem

FIGURE 8. — An excessively grazed area at the head of Parrish Creek. After the destruction of plant cover, these low-gradient slopes contributed heavily to the 1930 floods.

caused by the increasing impact of recreation is growing in importance. Overgrazing along trails and near campgrounds, concentrated pawing, trampling, and walking destroys vegetation, compacts the soil, reduces infiltration, and sets in motion both wind and water erosion. Promiscuous use of cross-country vehicles, especially "jeeps" and "scooters" or similar self-propelled wheeled vehicles, is creating erosion and sediment problems at an alarming rate.

Concerted effort is needed to quantify the damages and develop ameliorative measures for this problem that is rapidly being aggravated by increasing recreational pursuits.

Discussion

This synoptic consideration of selected data on sediment yields has highlighted a number of basic tenets, which, if ignored, can only lead to a continuance or acceleration in sedimentation; or which, given proper regard and application, can substantially reduce sediment yields resulting from the impact of land uses.

Soil is a product of climate and vegetation acting on geological parent material over a long period of time. Vegetation diminishes soil erosion and enhances soil stability. Vegetation may be destroyed purposely, wantonly, or accidentally. But irrespective of the manner in which vegetation is destroyed, the ultimate effects are identical. This conclusion is based on mounting volumes of evidence that vegetation is essential to the development, productivity, and stability of all soils.

Soils vary in their inherent resistance to erosion and their behavior or reaction to different uses. Some are more easily disturbed than others; some are more easily compacted than others; and some regain stability more readily than others. Therefore, protection must be shaped, designed, and applied in accord with the characteristics of the soil in question.

All land uses are potential precursors of sediment production. Abusive or excessive uses inevitably increase sediment yields. These must be guarded against because, ordinarily, natural recovery of stability by damaged watersheds often proceeds so slowly as to be unacceptable to proper land management.

Examples of sediment yields presented herein are necessarily limited. Yet, it is believed that they suffice to illustrate the delicate balance that exists between formative and disruptive forces. Little new information has been presented herein, but implications of the relations of various soil characteristics to sediment yields are sufficient to whet a more inquisitive pursuit of more specific and quantifiable relations. Although more is known of the effects of land use on sediment yields than is generally practiced, much yet remains to be learned to fill existing

voids. Logging methods need discovery or improving so that timber may be extracted from erodible areas without creating intolerable watershed damage. Roads must be better fitted to the topography to reduce the excessive cutting and filling now so characteristic of many logging roads. Disposal facilities need to be improved to reduce damages from water that concentrates on logging roads.

A little arithmetic illustrates the magnitude of the water disposal problem from roads. From a storm producing 1 inch of rain, a 12-foot road 1 mile long would intercept 39.5 thousand gallons of water. If the annual precipitation were 30 inches, the 1 mile of 12-foot width road would receive 21.5 thousand, 55-gallon barrels of water. Assuming a forest area with 10 miles of road per section, then 215,400 barrels of water would be intercepted. Because infiltration rates on road surfaces are low, most of this water soon becomes runoff — erosion occurs, sedimentation increases, and flood threats are heightened.

No one questions the need for continuing concerted effort to reduce destruction caused by wildfires. But with increasing use of prescribed fires, their effects on sediment production need to be determined for different soil-vegetation complexes.

Lastly, acceptable grazing intensities must be determined and rigorously applied to maintain soil stability on undamaged or rehabilitated rangelands and to promote stability on those now damaged.

We have progressed in understanding relations and associated problems between ecological factors, land uses, and sediment yields; yet, further refinements are needed. These will be forthcoming, but in the meantime a proper synthesis and application of knowledge now at hand would do much to reduce further the loss of soil and the production of sediment.

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SEASONAL DEBRIS MOVEMENT FROM STEEP MOUNTAINSIDE SLOPES IN SOUTHERN CALIFORNIA

[Paper No. 12]

By JAY S. KRAMMES, *research forester, Pacific Southwest Forest and Range Experiment Station, Forest Service, Glendora, Calif.*

Most people become concerned about flood-borne debris only when it causes property damage or is deposited in reservoirs. Much of the debris is scoured from stream channels, where it accumulates as the product of side-slope erosion. The interest in debris movement down mountain slopes, then, stems from flood and debris control problems downstream. Though debris produced during storm runoff is often the most spectacular, dry-season debris movement is an important part of the erosion in the San Gabriel Mountains. This paper reports a comparative study of dry- and wet-season debris movement in these steep, unstable watersheds.

A study in the Los Angeles River Watershed by Retzer and others [1] showed the sources and processes of debris movement. They mapped the sources of debris and determined the important source areas, in approximate descending order, to be "(1) streambanks and slopes rejuvenated by undercutting; (2) slopes with south exposures; (3) very steep slopes; (4) fault zones and steep fault faces; and (5) deep colluvial-alluvial deposits on slopes where undercut by roads or streams."

The active agents of erosion were observed to be gravity, water, wind, and the daily freeze-thaw cycle, the latter occurring primarily at high elevations. These agents can act alone, but they commonly work together, especially water and gravity.

Erosion took place mainly as granular movement on the surface rather than as deep-seated movements of soil masses. Granular debris moved as dry creep and as slope wash. Both of these processes were active over the entire study area. About 90 percent of the area mapped was affected by dry creep movement to some degree.

Dry creep movement is not found in most regions. It requires steep slopes with dry, loose material on the surface. The dry slide material ranges in size from larger rocks to fine soil particles. When the slopes are vegetated, some of the dry creep material is detained behind rocks, stumps, and brush. Other material not detained continues to move downslope and collect as cones in channels.

The slopes of the San Gabriel Mountains maintain a precarious equilibrium. The average slope of the land is more than 65 percent, or above the angle of repose for unconsolidated soil materials. Downslope movement may be triggered by slight disturbances: movement of animals, the wind, or earth tremors.

Surface Debris Movement Study

A study was started in 1953 to determine the cause (gravity or water), rates, and amounts of debris moving downslope and into channels.[2]

Study Sites

Nine study sites were located in the Los Angeles River Watershed. Five of these sites are on rejuvenated slopes.[3] The other four sites are on slopes not affected by rejuvenation (table 1).

TABLE 1.—*Characteristics of debris movement study sites*

Location (name and No.)	Slope condition	Aspect	Area	Cover density	Average slope
			Acres	*Percent*	*Percent*
Lower Brown —site 1	Rejuvenated	NE.	3.45	95	70
Lower Brown —site 2	do	NE.	3.36	95	70
Lower Brown —site 3	do	SE.	.72	65	90
Lower Brown —site 4	do	SE.	1.45	65	90
Lower Brown —site 5	do	SE.	.34	65	90
Upper Brown —site 6	Non-rejuvenated	SW.	.68	95	55
Falls Canyon —site 7	do	NE.	1.53	95	60
Singing Springs —site 8	do	SW.	1.30	65	60
Singing Springs —site 9	do	NE.	1.64	85	55

Study sites were located on generally north- or south-facing slopes that were undisturbed by fire or road building. The experimental sites were chosen in a single rock type — the Wilson Diorite, which underlies about one-third of the San Gabriel Mountains. The soils are character-

[1] RETZER, J. L., and others. PRELIMINARY REPORT, ORIGIN AND MOVEMENT OF SEDIMENTS IN THE LOS ANGELES RIVER WATERSHED, CALIFORNIA. U.S. Forest Service. 103 pp., 1952. [Typed.]

[2] ANDERSON, H. W., COLEMAN, G. B., and ZINKE, P. J. SUMMER SLIDES AND WINTER SCOUR . . . DRY-WET EROSION IN SOUTHERN CALIFORNIA MOUNTAINS. U.S. Forest Serv. Pacific Southwest Forest and Range Expt. Sta. Tech. Paper 36, 12 pp., illus. 1959.

[3] Rejuvenated slopes are the steep slopes flanking stream channels in which renewed channel downcutting has removed the toe of the slope, creating an unstable condition where active erosion is taking place.

istically shallow, coarse-textured, noncohesive, and very erodible.

Measurement of Debris

Debris moving downslope is caught in troughs that are connected to the original soil surface by a concrete apron. The troughs are built on the contour from a point of a ridge across a segment of the slope and end near a drainage channel. Troughs are placed across erosion chutes [4] at the rejuvenated slope study sites. Catchment troughs range in length from 10 to 431 feet. Four-foot high barriers are installed at several of the sites to catch bouncing rocks and excessive debris yields.

The material caught in each trough is removed and weighed, corrected for moisture, and sampled for organic matter and rock content.

Results

Debris Production From Unburned Slopes

Anderson, Coleman, and Zinke [5] reported the first 5 years' debris production from these sites under long unburned conditions. They found that the greatest source of debris was from the south-facing rejuvenated slopes. These sites yielded an average of 3.56 tons per acre per year — 5 to 10 times the average on other sites. Debris movement during the dry season exceed wet-season movement at most of the study sites. Even under unburned conditions, at least 0.2 ton per acre per year was measured at all study sites.

Rainfall during the first 4 years of the study was 77 percent of normal. The fifth year's precipitation was 143 percent of normal, but there were no high intensity storm periods. The first gentle rains increased the cohesion of the soil and tended to reduce wet-season debris movement.

Krammes [6] reported a sixth year's (1958–59) debris production under unburned conditions. The south-facing rejuvenated slopes were again the greatest producers. Precipitation during the season was 70 percent of normal. One storm caused the wet-season movement to exceed dry-season movement at the south-facing rejuvenated sites and the north nonrejuvenated site 9. Six years of debris production under unburned conditions appears in tables 2, 3, and 4.

Debris Production From Burned Slopes

In October 1959 a wildfire swept through seven of the nine study sites (sites 1 through 7). Debris movement began almost immediately after the fire passed.[6] The fire destroyed the low-growing brush that formerly detained de-

[4] Erosion chutes are caused by the concentrated movement of soil and rock down segments of steep slopes during both wet and dry periods. Reference: BLACKWELDER, ELIOT. THE PROCESS OF MOUNTAIN SCULPTURE BY ROLLING DEBRIS. Jour. Geomorphology 5(4): 324-328. 1942.

[5] See footnote 2.

[6] KRAMMES, JAY S. EROSION FROM MOUNTAIN SIDE SLOPES AFTER FIRE IN SOUTHERN CALIFORNIA. U.S. Forest Serv. Pacific Southwest Forest and Range Expt. Sta. Res. Note 171, 8 pp., illus. 1960.

TABLE 2.—*Debris production by seasons, south-facing rejuvenated slopes (Lower Brown Canyon)*

Period	Site 3			Site 4			Site 5			Average sites 3, 4, and 5		
	Season			Season			Season			Season		
	Dry	Wet	Yearly	Dry	Wet	Yearly	Dry	Wet	Yearly	Dry	Wet	Yearly
	Tons per acre			*Tons per acre*			*Tons per acre*			*Tons per acre*		
Prefire (1953–59)	0.84	1.86	2.70	0.82	0.83	1.65	1.99	0.89	2.88	1.27	1.42	2.69
Postfire												
1st year (1959–60)	7.62	2.58	10.20	28.69	3.26	28.95	23.38	.81	24.19	21.93	2.74	24.76
2d year (1960–61)	1.43	34.97	36.40	5.05	21.53	26.58	1.24	10.30	11.54	3.51	23.87	27.38
3d year (1960–62)	2.94	29.83	32.77	6.97	9.55	16.62	.....	.....	(1)	5.62	16.57	[2]22.19

[1] Trough destroyed by large boulder.
[2] On the basis of sites 3 and 4 only.

TABLE 3.—*Debris production by seasons, north-facing rejuvenated slopes, colluvial soils (Lower Brown Canyon)*

Period	Site 1			Site 2			Average sites 1 and 2		
	Season			Season			Season		
	Dry	Wet	Yearly	Dry	Wet	Yearly	Dry	Wet	Yearly
	Tons per acre			*Tons per acre*			*Tons per acre*		
Prefire (1953–59)	0.14	0.15	0.26	0.13	0.13	0.26	0.13	0.14	0.27
Postfire:									
1st year (1959–60)	4.35	.49	4.84	3.55	.23	3.78	3.95	.36	4.31
2d year (1960–61)	.23	1.40	1.63	.25	1.57	1.82	.24	1.48	1.72
3d year (1961–62)	.45	.82	1.27	.11	1.68	1.79	.28	1.25	1.53

TABLE 4.—*Debris production by season, south- and north-facing nonrejuvenated slopes (Upper Brown, Falls Canyon, and Singing Springs)*

Period	Site 6 south			Site 7 north			Site 8 south [1]			Site 9 north [1]		
	Season			Season			Season			Season		
	Dry	Wet	Yearly	Dry	Wet	Yearly	Dry	Wet	Yearly	Dry	Wet	Yearly
	Tons per acre			*Tons per acre*			*Tons per acre*			*Tons per acre*		
Prefire (1953–59) . . .	0.32	0.23	0.55	0.16	0.10	0.26	0.13	0.22	0.35	0.08	0.12	0.20
Postfire:												
1st year (1959–60).	.64	1.53	2.17	2.05	2.03	4.08	.03	.02	.05	.03	.01	.04
2d year (1960–61).	.11	4.48	4.59	.08	2.47	2.55	.04	.96	1.00	.04	.28	.32
3d year (1961–62).	.02	2.13	2.15	.01	1.28	1.29	.03	.46	.49	.03	.18	.21

[1] Vegetation unburned.

bris on the side slopes. Great quantities of debris moved downslope and into stream channels.

First Postfire Year

Dry-season debris movement in the first year after the fire ranged from 0.6 to 21.9 tons per acre (tables 2, 3, and 4). South-facing slopes flanking rejuvenated stream channels again had the highest annual production, more than 10 times the already high prefire rate (fig. 1). Nearly 90 percent of the debris came during the dry season.

North-facing rejuvenated slopes showed a postburn rate of 4.3 tons per acre per year, or an increase of about 16 times the unburned average (table 3). Debris production from the north nonrejuvenated site increased 16 times. The smallest increase (4-fold) occurred at the south nonrejuvenated site (table 4). Rock outcrops may have served to stabilize this site somewhat.

Precipitation during the first postfire year was 59 percent of normal and no high intensity storms were recorded.

The burned area, including the study sites, was seeded with annual ryegrass (*Lolium multiflorum*) and black mustard (*Brassica nigra*) after the fire. Because of the below-normal rainfall and extended dry periods during the wet season, the seeding was not successful. On the rejuvenated sites the seed moved downslope with the debris and was buried in the stream channel.

Second Postfire Year

Rainfall in the 1960–61 year, although only 35 percent of normal, contained four storms, totaling 7.35 inches of rainfall that produced debris movement. The south-facing rejuvenated sites, which were the most unstable, produced almost 27 tons per acre (table 2).

Most of the debris was moved in the second storm of the year. The highest 5-minute intensity recorded during that storm was 1.68 inches per hour. More wet-season debris was produced during this storm from all sites than had occurred during any of the previous 8 years of measurement. Wet-season debris production appears to be related more closely to intensity than total amount of storm rainfall.

Dry-season debris production in the second postfire year was considerably less than in the first postfire dry-season.

Third Postfire Year

Precipitation during the 1961–62 year was 96 percent of normal. The first three storms of the year, with a total of 5.62 inches of rainfall, produced more than 10 tons per acre at the south-facing rejuvenated sites. Another 5 tons per acre were measured in the rest of the wet season (almost 19 inches of rainfall). Wet-season debris movement exceeded dry-season movement. Dry-season debris movement was still considerably higher than the prefire rate on the south-facing rejuvenated sites but much less than the rate during the short period just after the fire (table 2).

Discussion

Debris that is eroded from the side slopes arrives in the channels through the action of wind, water, and gravity. However, these forces do not act equally or independently. There is always a gradual downslope movement of debris in the San Gabriel Mountains. The gradual soil movement during dry seasons is the "base flow" and the wet-season movement is analogous to "storm flow." Debris movement was separated into dry-season and wet-season movements to determine variations in rate between seasons. The first light rains of the wet season quite often have little or no effect on dry movement. As soil moisture increases, cohesiveness is given to the soil and dry movement slows down. This lasts as long as soil moisture is maintained. With additional rainfall, wet movement predominates. If prolonged dry periods between storms cause a reduction in surface moisture, dry creep begins again.

Over the years, many tons of debris are deposited in stream channels during both wet and dry seasons. This material remains poised in the channels and will be moved only when winter runoff has sufficient carrying power. Such flows occur on the average of once in every 5

to 6 years. Thus, side slope erosion provides much of the flood debris that is thought of as bank or channel scour. Stabilizing these steep mountain slopes remains as a challenge to future erosion-control efforts in the San Gabriel Mountains of southern California.

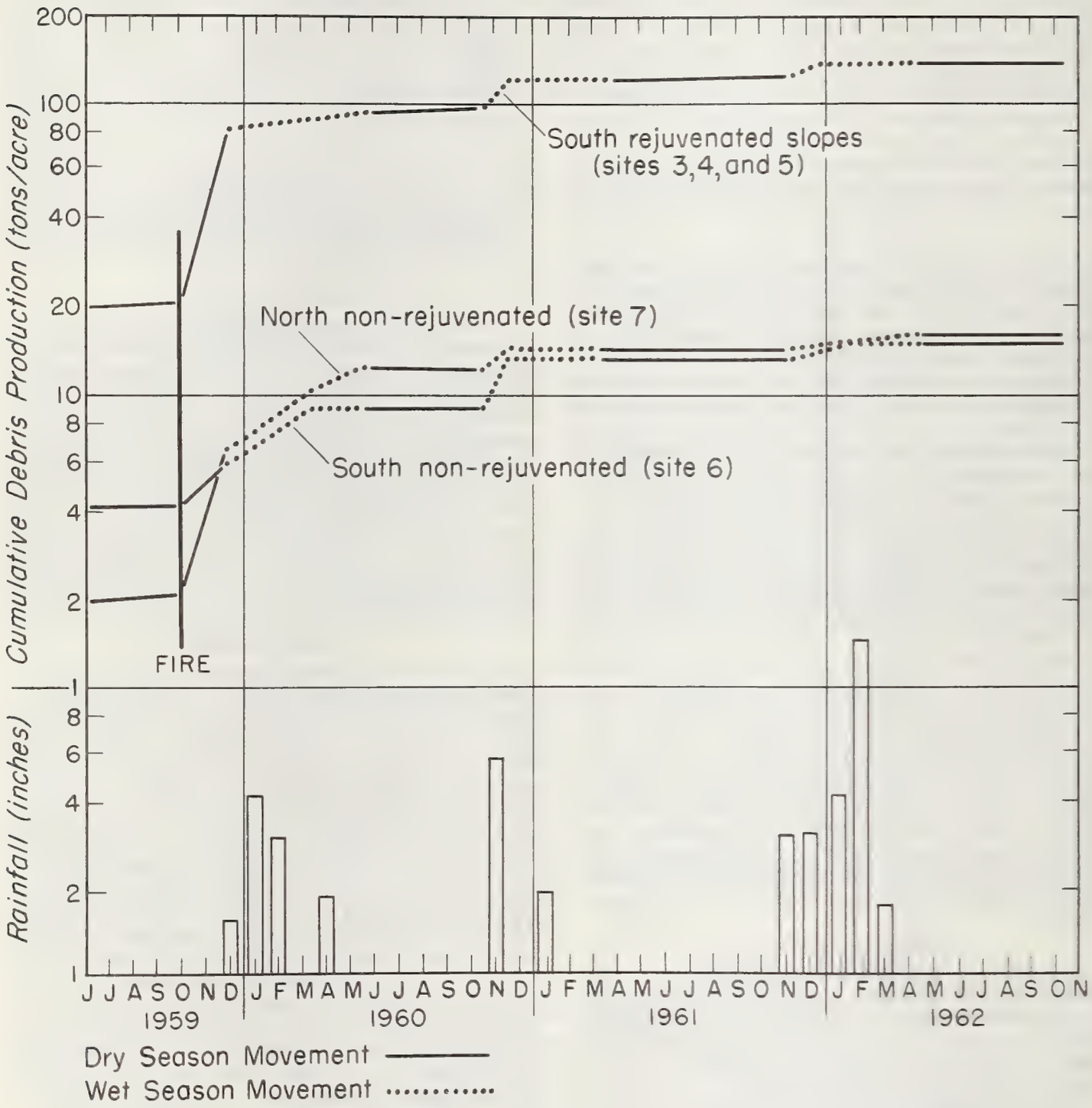

FIGURE 1. — Seasonal debris movement before and after fire in the Los Angeles watershed.

FUNCTION AND SIGNIFICANCE OF WIND IN SEDIMENTOLOGY

[Paper No. 13]

By W. S. CHEPIL,[1] *research investigations leader, Southern Plains Branch, Soil and Water Conservation Research Division, Agricultural Research Service* [2]

Abstract

The actions of wind on soil may be classed roughly into three categories: (1) Soil removal, (2) deposition, and (3) mixing. The wind's activity on soil is mostly in arid regions, but deposition extends even onto the humid regions.

Erosion of soil by wind includes three types of soil movement — saltation, surface creep, and suspension. Impacts of grains in saltation cause movements by surface creep and suspension. Dust carried in suspension is lifted by upward currents of turbulent wind. Once lifted off the ground, it is carried high in the air and is deposited in uniform layers far from its source. Fine dust may circle the earth many times. The composition of freshly deposited dust is like the composition of loess of the Pleistocene age. Estimates show that during the last 40 years an equivalent depth of 1.2 inches of soil material, on the average, has been removed from about 750,000 square miles of the Great Plains. In one semiarid portion of the Great Plains, an average of 9 inches of topsoil was removed from fields that were cultivated for about 20 years.

Removal of dust from one source is compensated by deposition elsewhere. Estimates have shown that during the last 20 years about one-half inch of atmospheric dust has been deposited in grassland near the wind-eroded region. Research is being undertaken to determine rates of deposition in humid regions of the United States.

Introduction

Unlike water, which tends to carry the soil from higher to lower elevations onto alluvial plains and into the sea, the wind scatters the fine soil constituents and deposits them in a uniform mantle over extensive areas near and far from their source. Removal of these fine soil materials from one area is therefore somewhat compensated by deposition at another. Movement also occurs in reverse so that considerable mixing of soils great distances apart occurs.

Removal is mostly from arid regions, but deposition and mixing extend imperceptibly, even to a casual observer, onto the humid regions.

Nature of Wind Erosion

Wind erosion is characterized by three types of soil movement — jumping (saltation), rolling and sliding (surface creep), and floating in the air (suspension). Impacts of grains in saltation cause movements by surface creep and suspension (*6*).

Saltation is caused by lift and drag forces against the surface. The saltating particles are the most erodible. They range from about 0.1 to 0.5 mm. in equivalent diameter (based on 2.65 density).

Particles too large to be moved by wind alone creep readily under impacts of saltation. They range from about 0.5 to 2 mm. in equivalent diameter, depending on wind velocity.

Contrary to general opinion, dust is highly resistant to erosion by direct pressure of the wind but is readily moved by impacts of larger particles moved in saltation. Once kicked up by saltation, it is carried by the upward currents of turbulent flow. Dust carried in suspension is generally less than 0.1 mm. in equivalent diameter. Dust clouds often rise 3,000 to 4,000 meters and are the most visible and therefore the most dramatic aspects of dust storms (fig. 1).

Abrasion

The impacts of saltation also cause clods and surface crust to disintegrate to small fragments, which in turn are moved by wind. The longer erosion continues and the more the wind shifts from different directions, the greater is the quantity of erodible material formed by abrasion and the higher the rate of erosion. The materials detached from clods and surface crust tend to accumulate in dunes or drifts toward the leeward side of fields and, if they are fine, to be carried far through the atmosphere. The smaller the detached dust particles, the farther they are transported by the wind (*6*).

Avalanching

The tendency for saltating grains to accumulate toward the lee of fields causes an increase of erosion (avalanching) for a distance of 500 yards or more before the maximum soil flow that a wind of a given velocity can sustain is reached. Often the maximum flow is not reached because the distance across the eroding field is limited. The rate of soil avalanching varies directly with soil erodibility; that is, the more erodible the soil, the greater is the rate of avalanching and the shorter the distance at which maximum rate of erosion (soil flow) is reached. By the same token, the more erodible the soil, the narrower the erodible field has to be to keep the rate of soil flow down to a tolerable limit. This limit is generally taken as 0.2-ton-per-rod-

[1] Died Sept. 6, 1963.

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FIGURE 1. — Great quantities of dust are moved long distances during periods of wind erosion. Black blizzards such as this are the most dramatic aspects of duststorms.

width-per-hour under a 40-mile-per-hour wind velocity for bare soil before cultivation in spring. This is one-tenth of the possible maximum intensity that would occur under the same conditions without restricted distance across an erodible field (*3*).

Sorting

The wind acts on many soils like a fanning mill on grain — removing the finer fractions and leaving the coarser ones behind (*2, 7*). Silt and clay are thus removed and lag sands and gravels are left behind. Over the years, this sorting action makes the soils progressively coarser in texture. Finally, nothing is left but the infertile, skeletal material forming shifting sand dunes and gravelly pavements. History of many old civilizations is a record of struggles against such deterioration of the land (*1, 11*).

Sorting occurs on soils developed from glacial till, residual material, mountain outwash, and sandy materials of various origins. The wind separates such materials into several distinct grades:

Residual soil materials. — Nonerodible clods and rocks that remain in place.

Lag sands, lag gravels, and lag soil aggregates. — Semierodible grains that are moved slowly by wind and deposited here and there on the surface of eroded areas. They are moved primarily by surface creep.

Sand and clay dunes. — Highly erodible grains that usually are not very far removed from an eroded area. Their movement is primarily in saltation.

Loess. — Dust, which once lifted off the ground by impacts of saltating grains, is carried high in the air and is deposited in uniform layers near and far beyond the dunes. Dust is carried in true suspension. The composition of freshly deposited dust is like the composition of the loess deposited in the Pleistocene age (*2, 13, 14, 16*). Huge deposits of loess in many parts of the world show the great importance of wind as a geologic mover of dust.

In some cases, wind erosion almost completely removes the surface soil. This nonselective removal by wind is associated with loess that was already sorted and deposited from the atmosphere during past geologic eras.

Soil Removal

In the southern High Plains of the United States, Daniel (*7*) studied the physical changes in soils under cultivation and accelerated wind erosion. His studies were based on a comparative difference in mechanical composition of virgin soils and drifted material from cultivated land. He found that soil materials carried by wind from coarse- and medium-textured soils and subsequently deposited in drifts (dunes)

contained on the average 38 percent less silt and clay and 29 percent more sand than the adjacent virgin soil. No direct comparison was made of the composition of virgin and adjacent cultivated soil.

In Canada, Doughty and coworkers (*8*) studied the physical and chemical changes in soils brought about by cultivation and erosion in the Brown, Chestnut, and Black soil zones on the Prairie region. Their study was based on direct comparison in physical and chemical composition between cultivated and adjacent virgin soils. They found that:

(1) The greatest loss of silt, clay, and organic matter due to cultivation occurred on extremely sandy soils and the least on clay soils.

(2) Many loamy sands lost virtually all silt and clay that they contained under virgin conditions less than 60 years previously. During dry periods, some of these soils changed to shifting dune sand and were abandoned. During wet periods, vegetation encroached and stabilized some of the active dunes.

(3) Sandy loams lost about 15 percent of their original silt and clay content in the top 4 inches of soil and gained a corresponding proportion of sand. Provided the same rate of selective removal continues in the future, these soils will turn to virtual sand dunes within 150 or fewer years of cultivation.

(4) The medium-textured glacial till loams and silt loams lost on the average 6.5 percent of their original silt and clay in the 4 inches of topsoil. At this rate of loss, about 500 to 1,000 years would be required for the surface soil material to change to dune sand.

Further studies of mechanical sorting of soil by wind in Kansas and Colorado (*2*) showed that on loess soils, i.e., soils derived from material originally deposited by wind during the Pleistocene period, wind erosion did not affect the texture of the drifts or the residual soil. But on soils derived from Permian sandstone, the drifts contained 65 percent more sand and 65 percent less silt and clay than the residual soil. The wind-eroded fields contained 17 percent more sand than the adjacent noneroded fields. Depth of sampling in all cases was 1 inch. This change in condition of the soil was a result of only two or three windstorms that occurred in 1 week when 0.85-inch depth of soil was removed from the eroded fields.

Daniel (*7*) and Chepil (*2*) found that removal of soil by wind, with or without any sorting, caused the general depletion of organic matter by virtue of some or all of the topsoil, in which organic matter is generally concentrated, being removed. Where sorting of soil material occurred, the damage to surface soil (to depth of cultivation) was two-fold: (1) Depletion of organic matter, and (2) removal of silt and clay. The drift soil (soil that has been moved about by the wind and deposited in drifts on or near eroded fields) contained 71 percent sand as compared with 38 percent in the top 4 inches of noneroded fields. This depletion of silt and clay added further to the hazard of wind erosion and to the problem of how to hold the remaining soil.

Another study on the effects of wind erosion was conducted in western Kansas in 1949 and 1950 on soils developed from loess (*4*). These soils belong to the Ulysses and Baca series. Some parts of this region had approximately 60 percent of the land in native sod before World War II. Progressive breaking of the sod for cultivated crops started in 1942 and was virtually completed in 1950. A unique opportunity, therefore, existed to determine possible changes in soils associated with time-after-breaking. Physical and chemical analyses were conducted on (1) 31 newly broken fields (broken between 1936 and 1948), (2) 31 intermediate (broken between 1939 and 1944), and (3) 30 old cultivated (broken before 1936). The study showed that:

(1) The old cultivated fields had little or no topsoil (A horizon) left. The average distance to the lime layer was 10 inches, whereas on newly broken land it was 19 inches. This means that 9 inches of soil were gone within an agricultural history of about 2 decades.

(2) There was no appreciable accumulation of drifted material. Much of it was apparently fine enough to be carried into the atmosphere the same way it arrived. Only slight quantities of sandy material were observed near an occasional temporary watercourse.

(3) On the old cultivated fields the B horizon constituted the surface soil. This soil contained more clay and less silt and sand than did the surface soil of the newly broken land (table 1).

(4) There was substantially less organic residue (stubble and straw) in old than in newly broken fields. Apparently, the productivity of the old cultivated fields had dropped considerably during the short period under cultivation. This resulted in greater exposure of the soil to wind and water erosion. However, the soil of the old cultivated fields was cloddier and contained more coarse (>0.5 mm.) water-stable aggregates than did the newly broken fields.

(5) Soil losses, as measured by wind tunnel tests (table 1), were smaller from old cultivated fields than from newly broken fields, despite the newly broken fields having more crop residues. It is evident that the greatest rate of removal of soil by wind would occur within a few dry years after breaking. The native grasses were

TABLE 1.—*Physical and chemical properties of surface soils at various periods after breaking of virgin sod in western Kansas*

Fields (number)	Years after breaking	Clay <0.002 mm.	Total organic matter	Nitrogen	Organic residue >1.19 mm.	Clods >0.84 mm.	Water-stable aggregates >0.5 mm.	Amount eroded in wind tunnel	Computed erodibility [1]
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Tons/acre*	*Tons/acre*
30.....	19	21.0	2.13	0.120	0.67	58.1	12.9	0.40	0.49
31.....	6	18.0	2.49	.129	.89	54.4	11.6	.46	.60
31.....	2½	17.0	2.53	.129	1.13	50.2	10.3	.64	.80
Level of significance		Differences necessary for significance at indicated levels							
1 percent.....		2.3	0.37	0.013	0.24	4.5	2.4	0.25	0.25
5 percent.....		1.7	.28	.010	.21	3.3	1.8	.19	.18

[1] Based on dry soil fractions > 0.84 mm. Data from Chepil, Englehorn, and Zingg (*4*).

effective in creating a loose, finely granulated soil structure — a structure highly susceptible to erosion by wind. However, the grasses were able to protect the soil from erosion. Accelerated erosion occurred only after the land had been denuded of vegetation by overburning, overpasturing, and overcultivation (*12*).

To trace the removal of soil beyond the eroded fields, Chepil and Woodruff (*5*) measured dust concentration in the air during duststorms. They found that the average rate of dust movement in western Kansas and eastern Colorado during the 1954 and 1955 duststorms was about 10,000 tons per hour per vertical square mile against the earth's surface.

Further analyses of the Chepil and Woodruff (*5*) data indicated that the total duration of the 1954 and 1955 duststorms at Dodge City, Kans., (which lies on the eastern outskirts of the severely wind-eroded area) was about 435 hours. This means that 4.35 million tons of dust per square mile perpendicular to wind direction moved past that location and presumably from the wind-eroded area during 1954 and 1955. If it is assumed that this region is 400 miles wide along the direction of wind and that an acre-foot of soil weighs 2,000 tons, 0.1-inch depth of soil emigrated during 1954 and 1955. From the 1922-61 weather records at Dodge City, the total duration of duststorms was estimated to be 5,200 hours. If the intensity of duststorms was assumed to be the same throughout the whole period, the net removal during the 40 years was 1.2 inches (3 cm.) of soil. This estimate would be greater or smaller if we assumed the region to be narrower or wider than 400 miles along the direction of wind and greater if the dust transported above 1 mile were considered. On this basis, it is estimated that 48 million acre-feet of soil were removed from the United States part of the Great Plains of about 750,000 square miles during the 40 years. Some dust might have been brought into the wind-eroded region, but because the region is practically surrounded by mountains and humid regions, the immigrated quantity was probably relatively small.

Deposition and Mixing

The Great Plains region's loss must have been compensated by some other regions' gain. It is reasonable to expect that considerable dust deposition must have occurred in the more humid areas, especially east of the wind-eroded region. Preliminary measurements indicated that between 5 and 10 tons of dust per acre per annum were deposited at Manhattan, Kans. (in the wet subhumid region) during 1954 and 1955 as compared to an average of about 8 tons per acre per annum removed from the Great Plains during the same years. Smaller deposition probably occurred east of Manhattan. No information is available on what these quantities might have been for areas farther east or west of the Great Plains. Such information would be exceedingly valuable in helping to determine rates of soil renewal in different regions.

Considerable information is available on the depth, composition, and distance from source of loess deposited in past geologic eras (*10, 15*). Some information on the composition and source of dust presently deposited from the atmosphere in different regions is also available (*2, 8, 14, 16*). Furthermore, Free (*9*) estimated from meager data that the mean annual deposit east of the Mississippi River at that time was not less than 0.01 inch (1.67 tons per acre).

Authentic records of distance that dust can travel are rare. Free (*9*) cited records that indicated that dust can travel thousands of miles from its source. One notable example is that of Australian duststorms reaching New Zealand, 1,500 miles distant. Another is that of dust originating in the Sahara and traveling over southern and central Europe to Germany and England, about 2,000 miles. During the 1930's, dust deposits presumed to have originated in the Great Plains were observed on the Atlantic seaboard, 1,500 miles away. No information appears to be available in the literature on probable quantities of dust that could be trapped and retained with suitable land management. Therefore, in 1954, Chepil [3] undertook to find

[3] Unpublished data.

out rates of accumulation of aeolian material on sand and loamy sand in southwestern Kansas as influenced by land management. On land that had been returned to grass in 1946, a distinct layer of dust was present. The average depth of the dust layer was 0.4 inch (table 2) and its composition was a loam containing 64 percent silt and clay. Some of the dust evidently worked its way down about 1 inch deep, probably from insect and rodent activity and tramping by livestock. It is evident that the grass trapped at least one-half inch of aeolian material in approximately 10 years. The nearest cultivated field was about a mile away. On this field there was no evidence of dust accumulation (table 2). Here, the sandiest soil was the surface one-half inch. The surface soil evidently had been losing the finer mechanical constituents as fast or faster than it had been gaining them.

TABLE 2.—*Average composition and depth of atmospheric dust deposited in grass in western Kansas during 1946–56*

Site description	Depth	Mechanical composition in 1956		
		Sand >0.05 mm.	Silt 0.05–0.002 mm.	Clay <0.002 mm.
	Inches	*Percent*	*Percent*	*Percent*
Good grass cover established on abandoned cultivated land in 1946	0–0.4	35.6	40.3	24.1
	0.4–0.9	70.0	22.2	7.8
	0.9–1.9	89.7	7.1	3.2
	1.9–3.9	82.8	11.7	5.5
	3.9–5.9	82.0	10.8	7.2
Land cultivated primarily to grain sorghum since 1935	0–0.5	85.1	10.6	4.4
	0.5–1.0	86.4	9.4	4.5
	1.0–2.0	82.4	11.2	6.4
	2.0–4.0	77.3	15.7	7.4
	4.0–6.0	77.8	14.8	7.4

It is evident from this study that a fine soil texture, comparable to that of loess, can be regained slowly under grass on sandy soils in dry regions such as western Kansas if wind erosion on surrounding cultivated lands continues at the rate it has in the recent past.

Conclusions

The greatest damage from wind erosion has been the removal of fine constituents from soils containing a certain proportion of sand. This sorting action is apparently responsible for the formation of sandy wastes in dry regions.

On loess soils there is no such sorting action; the whole soil is removed bodily by the wind. The degree of damage to soil here is governed primarily by the depth of soil material.

The rate of soil removal since the breaking of virgin sod in dry regions has been enormous when considered in terms of the probable time required for soil material to be formed or laid down. Some of the stable grassland has been changed to shifting dunes within a generation.

Available data indicate that some of the dust removed from the Great Plains has been deposited in a thin mantle all the way east to the Atlantic Ocean and beyond. The deposited dust was probably the first to be removed by sheet erosion into streams, alluvial plains, and into the sea. Accumulations of loess east of the Great Plains apparently occurred only where the rate of deposition resulting from wind erosion exceeded the rate of removal, mostly by water erosion.

The extensive damage from erosion in dryland regions has been due to lack of adjustment between methods of land use and environment. Extensive grain growing has been one of the primary factors contributing to erosion. Why? Because greater income can be derived from grain than from utilizing native grass. Agricultural use of the land, therefore, has been developed largely on the basis of immediate rather than permanent returns from the land. This has resulted in considerable soil dissipation. Future population (if and when it expands beyond what the land will easily support) will pay for this dissipation.

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CAUSES OF VARIATIONS IN RUNOFF AND SEDIMENT YIELD FROM SMALL DRAINAGE BASINS IN WESTERN COLORADO

[Paper No. 14]

By GREGG C. LUSBY, *hydraulic engineer, U.S. Geological Survey, Denver*

Abstract

During a study of the effects of grazing on runoff, sediment yield, vegetation, and infiltration rates in paired, grazed and ungrazed, drainage basins in the Badger Wash area, it was determined that runoff and sediment yield were considerably less in the ungrazed basin of each pair, although no large changes in composition or density of plant cover were recorded.

A study of erosion on the hillslopes at Badger Wash revealed a seasonal cycle of loosening and compaction of the soil. The soil is loosened by frost action and then is compacted by rainbeat during the spring and summer. In grazed basins trampling by livestock causes an earlier and more pronounced compaction of the loosened soil, but in ungrazed basins the soil remains loose for a longer period of time and greater infiltration occurs. Therefore, the grazed basins yield higher rates of runoff and sediment yield, despite negligible changes in vegetational cover, because of the longer period of compaction resulting from livestock trampling.

Tentative conclusions on the basis of the first 5 years of record indicate that grazing exclusion, although not responsible for immediate major changes in plant density, does affect the rate of runoff and sediment yield.

Introduction

Much of the Western United States is arid to semiarid, and very little is known about factors governing erosion and runoff. With the abundance of more productive land in humid areas, the need for information on the semiarid and arid lands was not particularly urgent on a national scale. As a fuller understanding of utilization of the national resources was gained, it became apparent that large areas, which were sometimes thought of as "not worth spending money on," might eventually have to be utilized to the fullest extent possible. In order to formulate plans for proper utilization of semiarid rangeland, something must be known of the processes involved in the hydrology of these lands.

Much of the Colorado Plateau is semiarid-to-arid rangeland. Extensive areas underlain by shale are characterized by scant vegetation and high rates of runoff and sediment yield. Much discussion has taken place regarding the former condition of the forage on this range, and the statement is frequently heard that the land once supported considerably more vegetation than it does at the present time.

In 1953 several Federal agencies selected the Badger Wash drainage basin in western Colorado as the site of an intensive investigation of effects of grazing on runoff, sediment yield, vegetation, and infiltration rates. These data are collected as an aid in establishing criteria for land treatment practices. The Badger Wash basin, the location of which is shown in figure 1, is typical of a large part of the Colorado Plateau underlain by Mancos shale, which comprises a major outcrop area extending westward from Grand Junction, Colo., at the base of the Book Cliffs. Federal agencies originally involved in the study were Geological Survey, Forest Service, Bureau of Land Management, and Bureau

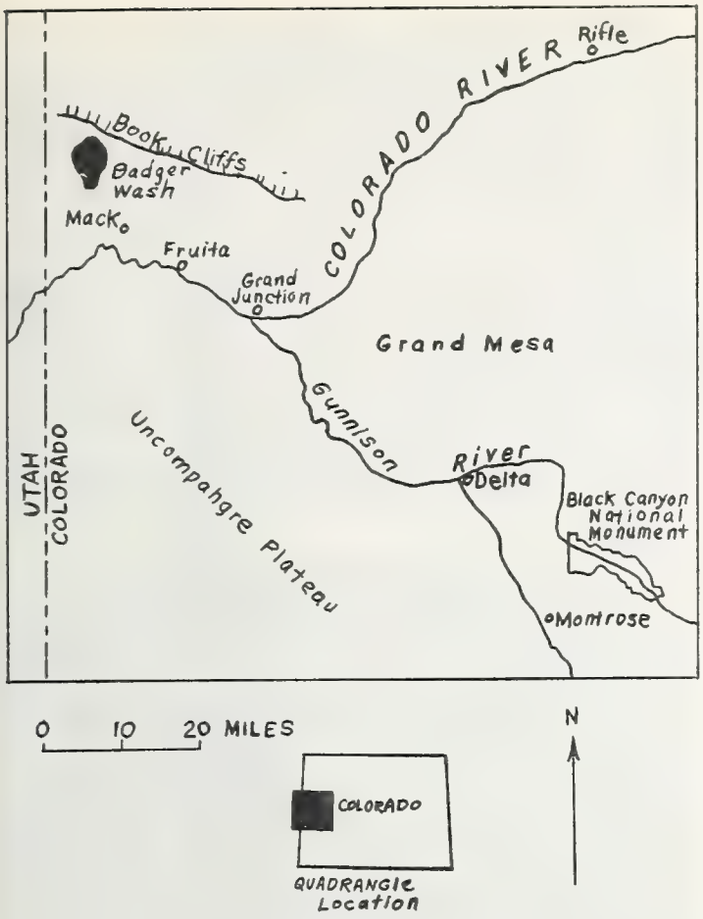

FIGURE 1. — Index map showing location of Badger Wash.

of Reclamation. In 1955 the Fish and Wildlife Service began the investigation of small mammal populations. A detailed description of the complete Badger Wash study may be obtained from the project report.[1]

The Geological Survey is concerned primarily with the measurement of precipitation, runoff, and sediment yield. In 1958 a study of hillslope morphology and erosion was begun by S. A. Schumm of the Geological Survey, and this study provided data that helped explain some characteristics of the runoff data. This paper combines some aspects of the original Badger Wash study and the hillslope study.[2] [3]

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Geology and Topography

The Badger Wash area is underlain by Mancos shale of late Cretaceous age. The shale is of marine origin and is highly saline. Thin sandstone beds are present at scattered locations and the weathering of these sandstone beds produces areas of sandy soils. The shale weathers to form a thin mantle of soil material overlying a zone of shale fragments on top of the bedrock. The weathered mantle is generally not more than 1 foot thick. Alluvial material is quite limited in areal extent, being restricted mainly to small areas along the larger stream channels.

Climate

The Badger Wash area is generally classified as arid, although according to Thornthwaite[4] the climate is semiarid about 50 percent of the time. Average annual rainfall at Fruita, Colo., located 16 miles southeast of the study area, is 8.3 inches, but yearly amounts range widely about this average. Temperatures range from below 0° F. to above 100° F. during most years. Potential evaporation rates in the area are high. Evaporation from a U.S. Weather Bureau Class A pan at Grand Junction averages about 90 inches for the months April to October.

Methods of Investigation

Runoff and sediment yield at Badger Wash are measured in reservoirs at the lower end of 8 paired drainage basins ranging in size from 12 to 101 acres. Drainage basins were chosen in matched pairs so that each pair was as nearly similar as possible with respect to size, soil, vegetation, topography, and aspect. One basin of each pair was fenced to exclude grazing. Reservoirs are equipped with water-stage recorders for measurement of inflow. Sediment yield is computed from periodic topographic surveys of the reservoir.

Precipitation is measured in nine recording rain gages located in the eight drainage basins, and rainfall amounts are computed for each basin by the Thiessen polygon method.

Runoff and Sediment Yields

During the period 1954–61 runoff from ungrazed basins averaged about 80 percent of that from the grazed basins (fig. 2) and ranged from 69 to 89 percent in the four pairs. A difference in sediment yield was measured that was even more pronounced than the change in runoff, with amounts ranging from 18 to 54 percent less in the ungrazed areas. The reduction of sediment yield did not appear to be directly correlated with reduction of runoff. In fact, the pair of basins that showed the greatest difference in runoff showed the least difference in

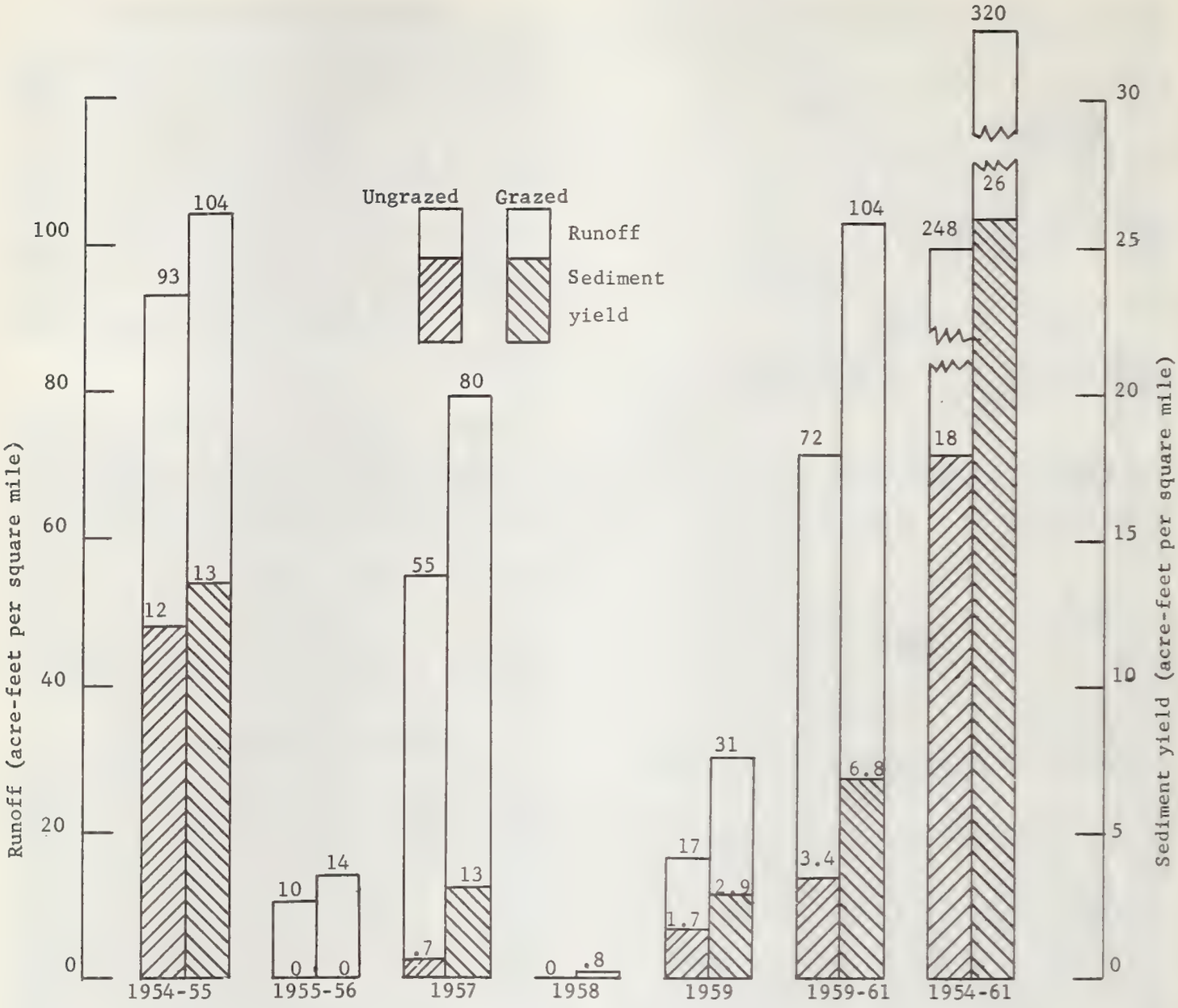

FIGURE 2. — Runoff and sediment yield — grazed and ungrazed areas.

sediment yield. According to Turner,[5] at the end of the first 5 years of the period, vegetation had not increased appreciably over that at the beginning of the study. If this is the case, some other factor besides vegetation density must be responsible for the change in runoff amounts.

In the study of hillslope erosion Schumm [6] described a cycle of heaving and compaction of the soil surface on Mancos shale hillslopes. This cycle is summarized briefly as follows. After a summer storm the soil surface is sealed and wet. As the soil is dried, a network of fine dessication fractures, which close when wetted, are formed. In this condition the hillslopes are relatively impermeable, and slopewash and rilling occur. During cycles of freeze and thaw during the winter, heaving takes place that destroys the rill patterns, loosens the soil, increases permeability and causes downslope creep of the Lithosol. When rain occurs during the next spring and early summer the soil is highly permeable and is able to absorb more rainfall than later in the year. As more rain falls during the summer, the soil again becomes compacted and infiltration rates are lowered. This cycle is illustrated in figure 3.

The occurrence of this cycle is borne out by analysis of precipitation and runoff records in Badger Wash. All storms greater than 0.10 inch during the period 1954–61 were arranged by

[5] See footnote 1.
[6] See footnote 2.

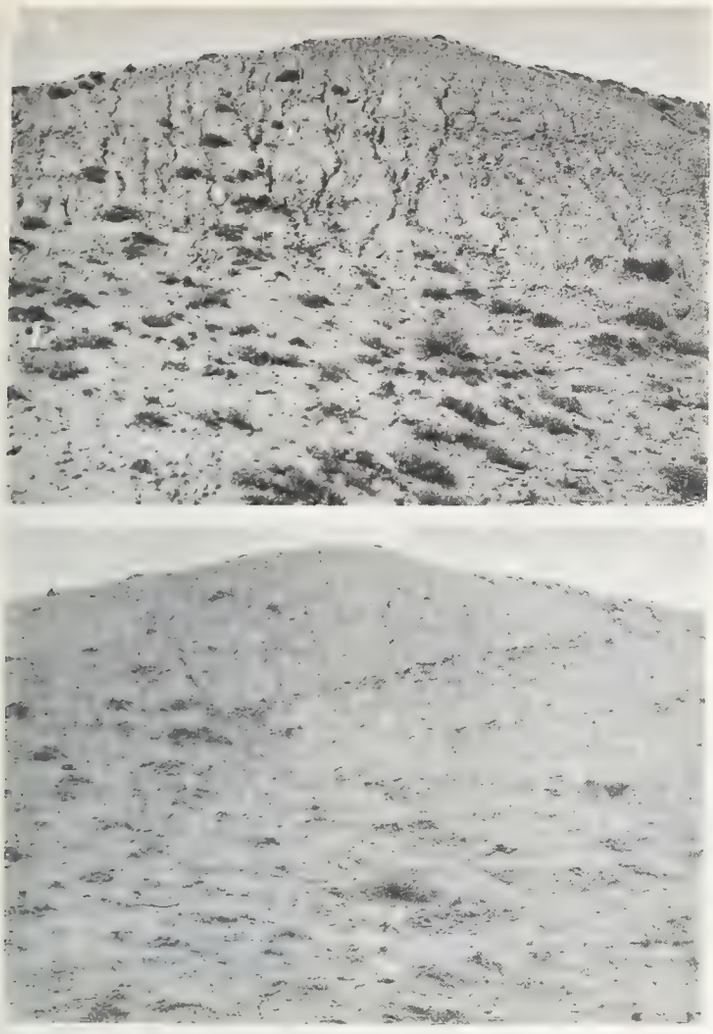

FIGURE 3. — Hillslope at Badger Wash, showing rilled surface in August 1959 (top) and same slope showing unrilled surface with some tracking in March 1961 (bottom).

0.10-inch increments and compared with runoff during two periods, April–June and August–October. As shown in table 1, runoff was greater during the fall period for all classes of storm.

Although the cycle of freeze and thaw apparently takes place in all drainage basins at Badger Wash, the effect on runoff appears to be greater in certain areas than in others. One of the pairs of drainage basins is located in an area of considerably rougher topography than the other basins. This area is composed of extremely steep slopes, a low density of vegetative cover, generally shaly soil mantle, and high drainage density. As generally considered, this area would appear likely to produce more runoff than other basins being studied, but such is not the case. Actually runoff amounts from this area have been about the same or slightly less than from the other basins, although sediment yield is considerably more. This anomaly is explained by differences in the cycle of soil

[7] See footnote 1.

TABLE 1.—*Spring and late-summer precipitation and runoff at Badger Wash*[1]

Range of storm precipitation (inches)	Period	Storms	Average precipitation	Average runoff	Runoff-precipitation
		Number	*Inches*	*Inch*	*Ratio*
0.10–0.20	Apr.–June	13	0.15	0.001	0.007
	Aug.–Oct.	23	.15	.008	.050
0.21–0.30	Apr.–June	4	.26	0	0
	Aug.–Oct.	11	.27	.039	.145
0.31–0.40	Apr.–June	3	.32	.007	.022
	Aug.–Oct.	11	.34	.063	.185
0.41–0.50	Apr.–June	3	.47	.020	.043
	Aug.–Oct.	7	.47	.093	.198
0.51–0.60	Apr.–June	1	.58	.120	.207
	Aug.–Oct.	2	.56	.180	.322
0.61–0.70	Apr.–June	1	.69	.004	.006
	Aug.–Oct.	4	.66	.290	.440
0.91–1.0	Apr.–June	0	.....	.....	.....
	Aug.–Oct.	1	.94	.380	.400
1.31–1.40	Apr.–June	0	.....	.....	.....
	Aug.–Oct.	1	1.34	.470	.350
All storms, 0.10–1.40	Apr.–June	25	.26	.004	.015
	Aug.–Oct.	61	.34	.079	.232

[1] From reference given in footnote 3.

expansion and compaction among basins. This swelling does not take place to as great an extent in the sandier soils as it does on the shaly soils. Although runoff may not be any greater from these steep shaly slopes than from other basins in the area, the sediment yield has been more because of the large supply of material made available for transport by frost action during the winter.

As shown previously, runoff and sediment yield have been considerably less from ungrazed areas than from grazed areas even though vegetation has not changed appreciably from area to area. Normally both cattle and sheep are grazed at Badger Wash from about November 16 until May 15. The trampling effect of grazing animals is quite apparent in the area as shown in figure 4. Undoubtedly the period from March 1 to May 15 is the period during the year when the top few inches of soil would be most adversely affected by trampling. Early in the spring the soil surface is in a loosened condition, and for a short period it contains enough moisture to provide maximum compactability.

Thompson [7] obtained penetrometer readings of the top 5 inches of soil in connection with the 1958 infiltrometer runs at Badger Wash. These tests were made in the infiltrometer plots after the wet run at each site and were completed during August and September. Unfortunately, penetrometer readings were not obtained during the 1953–54 seasons, but results of the 1958 tests show that significantly more force was required to penetrate the top inch of soil in the grazed basins than in the ungrazed basins. Also the difference in readings between paired basins was greatest in the basins underlain by sand-

FIGURE 4. — Hillslope at Badger Wash, showing effect of trailing by livestock.

stone and this difference appeared to persist into the second and third inch of soil. This difference may have been caused by one or a combination of factors about which at this time we may only speculate. The sandstone soils are apparently less subject to loosening by freeze and thaw than shale soils and could therefore have more carryover effect of compaction by trampling from year to year. Also, because of the nature of the topography and better inherent vegetation on the sandstone soils, these areas may have received more prolonged use than shaly areas. In any case, consideration of the penetrometer data would indicate that the upper part of the soil mantle is affected by grazing animals. The results are a higher runoff and sediment yield from the grazed areas, in spite of no large vegetational change.

Conclusions

Runoff from steep, highly dissected areas at Badger wash has been about the same as that from other less dissected areas of gentler slope. This is probably caused by loosening of the soil by frost action, which is dominant in steep shaly areas, thus making the soil more permeable. Although runoff has been about the same for the steep shaly areas and gentler sandy areas, sediment yield has been considerably more from the shale area. This indicates that sediment concentrations in runoff are much greater in these rough areas, which is a result of frost action making slopes unstable.

Runoff and sediment yields from paired drainage basins during the 8-year period 1954–61 were also determined to be materially affected by grazing animals. Although the loosening action of frost takes place in both grazed and ungrazed basins, the presence of livestock in grazed basins during the early spring period tends to cause compaction of the soil by trampling, thereby causing more runoff even though density and character of vegetation had not changed significantly during the first 5 years of the period.

THE ROLE OF THE LARGE STORM AS A SEDIMENT CONTRIBUTOR[1]

[Paper No. 15]

By ROBERT F. PIEST, *hydraulic engineer, USDA Sedimentation Laboratory, Soil and Water Conservation Research Division, Agricultural Research Service*

Introduction

A knowledge of the sediment contribution of large rainstorms, relative to the total quantity of sediment removed from a watershed over a long period of time, is a prerequisite to the planning of an efficient soil conservation program. Because the large storm has especially high flow rates and other erosive features that are apparent to even the casual observer and that are of great concern to technicians engaged in the design of soil conservation structures, its role as a sediment contributor is not always seen in proper perspective.

Wischmeier,[2] in an extensive study of soil losses from small, cultivated, single-cover plots at seven locations throughout the country, found that three-fourths of the soil losses were caused by an average of about four storms per year. Also, in a study of storm classes, he indicated that about one-third of the total soil losses was due to extreme storms having return periods greater than 2 years; one-third was due to moderate storms with return periods between 1 and 2 years; and another third was due to smaller storms with return periods shorter than 1 year. The bulk of soil loss on small cultivated plots, therefore, can be attributed to the more numerous storms that have at least a 50-percent probability of occurrence in any given year.

The purpose of this study is to evaluate the role of large storms in causing soil losses from small mixed-cover watersheds that range in size from 100 to about 100,000 acres.

Study Procedures

To attain the study objectives, it was necessary to estimate runoff and sediment yields for

[1] Cooperative research with Mississippi State University and University of Mississippi.

[2] WISCHMEIER, W. H. STORMS AND SOIL CONSERVATION. Jour. Soil and Water Conserv. 17 (3). 1962.

all storms, their frequency of occurrence, and the average annual runoff and sediment yields from each of the 72 watersheds studied. Suspended-sediment records seemed to offer the best approach to this study for watersheds in the size range under consideration.

A review of suspended-sediment bibliographies revealed that relatively little field data had been collected, until recently, from small watersheds. Records were most often available in daily summary form, although the tabulations of runoff and sediment yield by storm were available in a few cases. For the best sampled watersheds, the record lengths did not exceed 5 to 8 years; this was not enough to serve as the sole basis for predicting long-term sediment yield trends or for accurate determination of storm magnitude-frequency relation.

The average annual (long-term) yields upon which these storm studies were based were derived from the above-mentioned fragmental daily-runoff and suspended-sediment records and from long-term runoff records of watersheds in the vicinity. These yields are the subject of a paper.[3] The calculations of runoff and sediment quantities for large storms for the most part are based upon the premise that these storm quantities and durations are closely represented by daily quantities. Some of the assumptions inherent in the use of daily records to arrive at storm yields — the definition of a storm and some of the problems involved in assigning a return period to a storm of a specific size — are now discussed.

The Storm

A storm, unless specifically qualified, is generally understood to be a disturbance of the ordinary average conditions of the atmosphere, which may include any or all meteorological disturbances, such as wind, rain, snow, hail, etc. In this study we are concerned with the runoff-sediment producing aspects of the storm; most major runoff-producing events are rainstorms.

The duration of a storm has been variously defined. For studies on fractional-acre plots, it is considered by the ARS National Runoff and Soil Loss Data Laboratory to be ended when less than 0.05 inch of precipitation occurs during a period of 6 consecutive hours. To be consistent, for watersheds in the size range we are considering, the minimum duration of a storm should be approximately that time which will allow for practical hydrograph separation of discrete storms on the larger watersheds. This is usually less than 24 hours. Also, on the larger watersheds, 6 hours is about the minimum time between storms that will allow, with fair accuracy, hydrograph separation of successive storms.

Considering the nature of the data we are analyzing, which is usually published in daily summary form, and the runoff characteristics of watersheds in the 100- to 100,000-acre size range, the most consistent definition of a storm would be that it is a precipitation event, usually rainfall, that has a practical runoff duration of less than 24 hours.

This definition fits nearly all conditions that can be anticipated. For example, a sharp thunderstorm which produces a single runoff event on 100- and 100,000-acre watersheds will typically have practical runoff durations of less than 3 hours and 15 hours, respectively. (That is, more than 95 percent of the runoff and sediment load will pass from basin in that time, although the remainder may require several days.)

Due to these considerations, runoff volumes for watersheds in the size range herein considered are approximately the same whether reported on a daily or a storm basis.

The Flow Duration Curve

Experience with flow duration data indicates that, unless extremely unusual meteorological conditions prevail, a stream-gaging record of relatively short duration will closely approximate long-term conditions for low and moderate discharges. Only the high discharge portion of the short-term flow duration curve is subject to any great change.

Also, a daily flow duration curve is very similar to a duration curve that is based upon storm volumes if the duration of each storm is considered to be a day. Figure 1 shows typical variation between "storm" and daily flow duration curves for a 1.76- and a 117-square-mile watershed in northern Mississippi. The duration curves for a given watershed are very similar, but for very high discharges the "storm" duration curve values are slightly higher. (The divergence of the low-discharge parts of the curves is accented here by the fact that these streams are not ephemeral, but this divergence is irrelevant to a report concerned with sediment yields.)

From these and other data, the author has concluded that the high discharge part of the "storm" flow duration curve has somewhat greater values for a given duration than the daily flow duration curve. This finding is logical, in view of the fact that a few storms have durations greater than 24 hours, whereas others occur before and after midnight and partial storm volumes are therefore reported over several days.

A corrective procedure has been used that will allow the daily flow duration curve to

[3] PIEST, R. F. LONG-TERM SEDIMENT YIELDS FROM SMALL WATERSHEDS. For presentation at meeting of Internatl. Union of Geodesy and Geophysics, Berkeley, Calif. August 19-31. 1963.

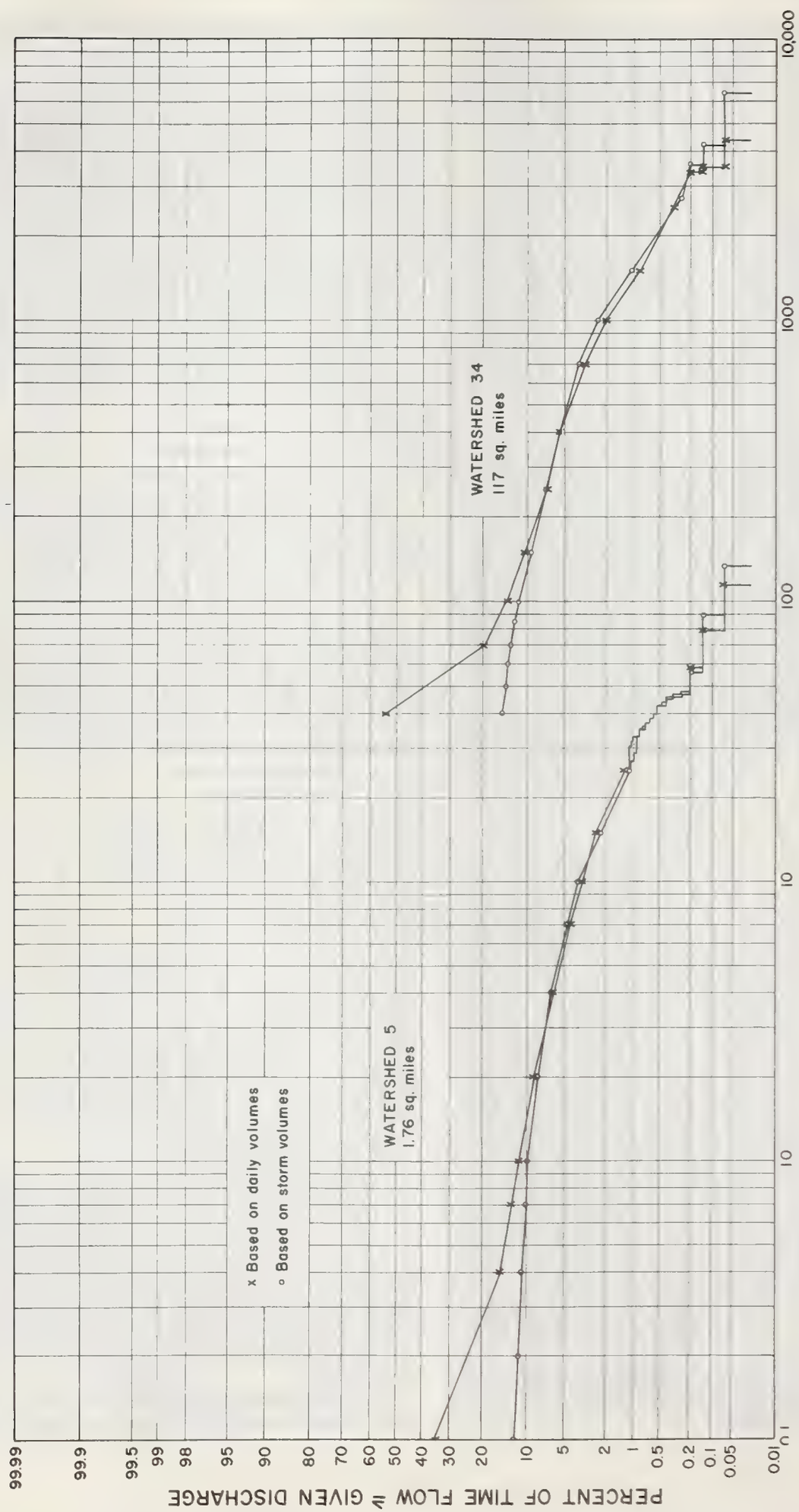

FIGURE 1. — Daily versus storm flow-duration curves for Pigeon Roost Creek watersheds, January 1957–December 1960.

approximate the "storm" flow duration more accurately. If a known relation between rainfall parameters and runoff volumes exists (such as a rainfall-runoff regression) or if data are available that allow graphical rainfall-runoff relations for short periods of record to be extrapolated on the basis of Weather Bureau data on extreme rainstorms, then computed estimates of runoff volumes for large storms with known return periods will verify a daily flow duration curve for use in storm analyses or will indicate needed adjustment.

A partial duration series of storm volumes is also a helpful check on the daily flow duration curve, although records are usually so short that limited confidence can be placed in this procedure.

The Water-Sediment Relation

The average daily water-sediment relation, as defined by short-term records for each of the watersheds, was extrapolated to represent long-term conditions by several procedures. The principal extrapolation assumption was that, at some high runoff volume, the concentration would reach a maximum and subsequent higher storm runoff volumes would contain sediment concentrations, on the average, that did not vary appreciably from this maximum. (Experience indicated, however, that this assumption did not apply for a few watersheds that were light sediment contributors.) The representative long-term water-sediment curve was then utilized with the long-term flow duration curve to construct a sediment duration curve.

As with daily flow duration curves, we have assumed that there is little practical difference between the water-sediment relation, whether on a storm or daily basis. Figures 2 through 5 compare daily and storm water-sediment relations for a large and a small watershed in Pigeon Roost Creek Basin. The middle and upper discharge segments of the curve are nearly identical, although there is evidence that a storm curve has slightly lower sediment discharges, at comparable runoff rates, than the daily curve.

Computations

The 4-year daily record of runoff for watershed 32, Pigeon Roost Creek Basin, Mississippi, was extrapolated on the basis of index-station [4] comparisons. By this procedure, runoff records were generally compared with a minimum of two long-term records for streams in the vicinity. The extrapolated daily flow duration curve $[q_w = f(t)]$, with time as independent variable, is plotted against the daily mean discharge rate (q_w), in cubic feet per second, as the dependent variable (fig. 6). If the dimensionless time scale is assumed representative of a given long-term period (Y), in years, this variate can be converted into (t) days, from 0 to 365.25 Y, and the area under the curve, in cubic feet per second-days, is then the total runoff volume (Q_w) for the long term period.

$$Q_w = \int_0^{365.25\,Y} f(t)\,dt$$

The percentage (P) of the total runoff volume that can be attributed to the highest m days (or storms) of record is then

$$P = \frac{\int_0^{m} f(t)\,dt}{\int_0^{365.25\,Y} f(t)\,dt} \times 100$$

In figure 6 the large crosshatched area represents the 50 highest ranked events in 100 years.

In practice, the runoff volumes are best computed by use of Simpson's rule or by graphic approximation.

A sediment duration curve can be constructed similarly from the flow duration information of figure 6 and knowledge of the average long-term water-sediment relation (sediment rating curve). The high sediment discharge part of a sediment duration curve is reproduced in figure 7, where the abscissa values are in terms of ranked events, each having a duration of 1 day rather than percentage of time.

Summary and Conclusions

Table 1 summarizes the relative sediment contributions of storms of various sizes from 72 small watersheds in 17 States. The watershed sediment yield caused by large storms (with a return period greater than 2 years) varied from 3 to 46 percent of total suspended-sediment yield; the yield from moderate storms with a 1- to 2-year return period ranged from 3 to 22 percent of the total; storms with a return period of less than 1 year were the cause of 34 to 92 percent of total suspended-sediment yield.

For most watersheds more than one-half of the soil losses are attributable to the smaller storms that occur more often than once a year, on the average. We can therefore expect that recommended land use and land treatment practices in upland areas, i. e., those elementary conservation measures that do not cost much and can be readily renewed after damage from major storms, would result in significant reduction in downstream sedimentation. In some problem areas, they would be sufficient to sustain the design life of a reservoir without the aid of expensive auxiliary structures.

The accuracy of computations is not sufficient to discern many trends, but it is clear that

[4] SEARCY, J. K. WATER SUPPLY PAPER 1542A. U.S. Geological Survey. 1959.

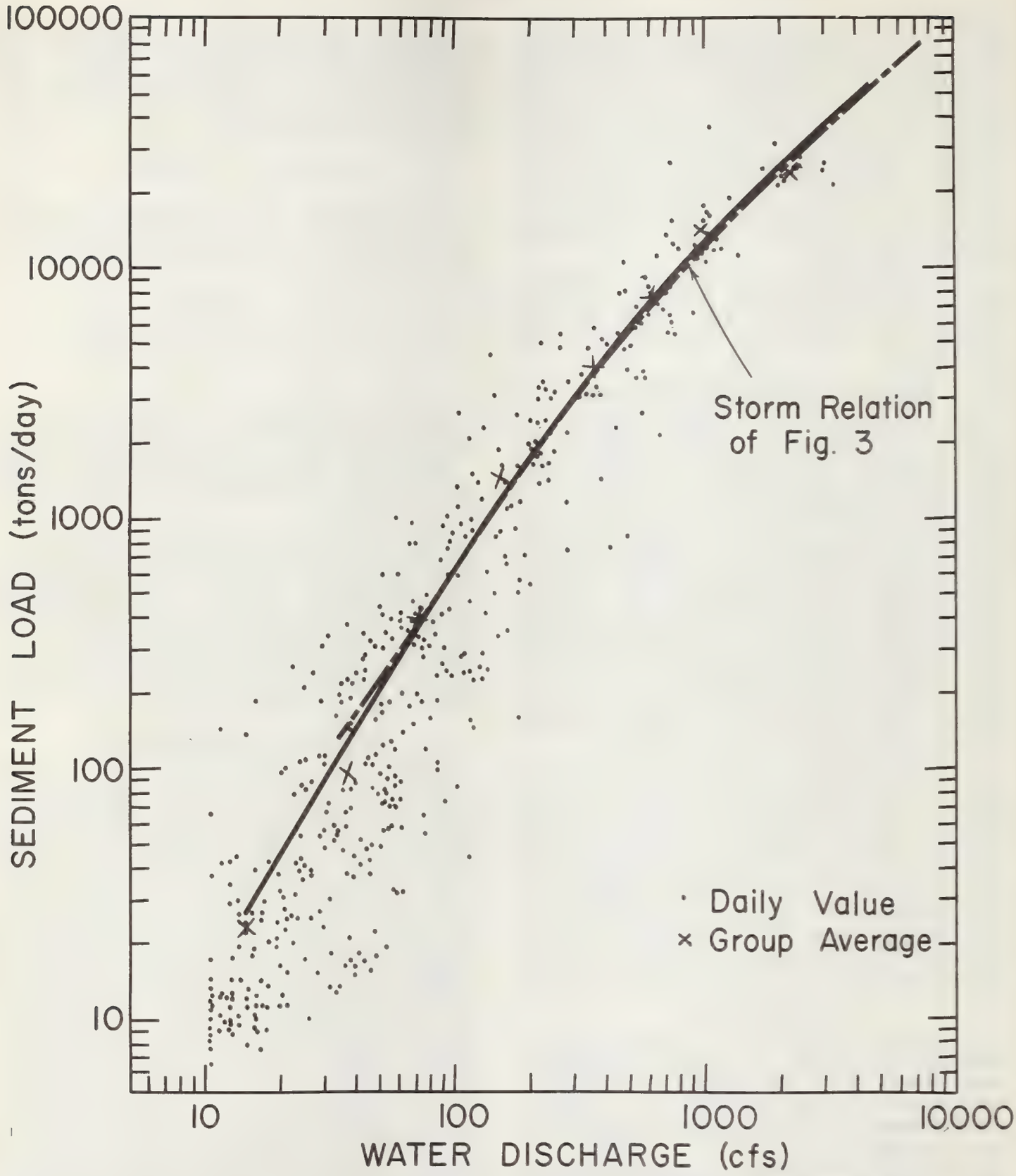

FIGURE 2. — Direct runoff versus sediment discharge, by day, Pigeon Roost Creek Watershed 34, January 1957–December 1960.

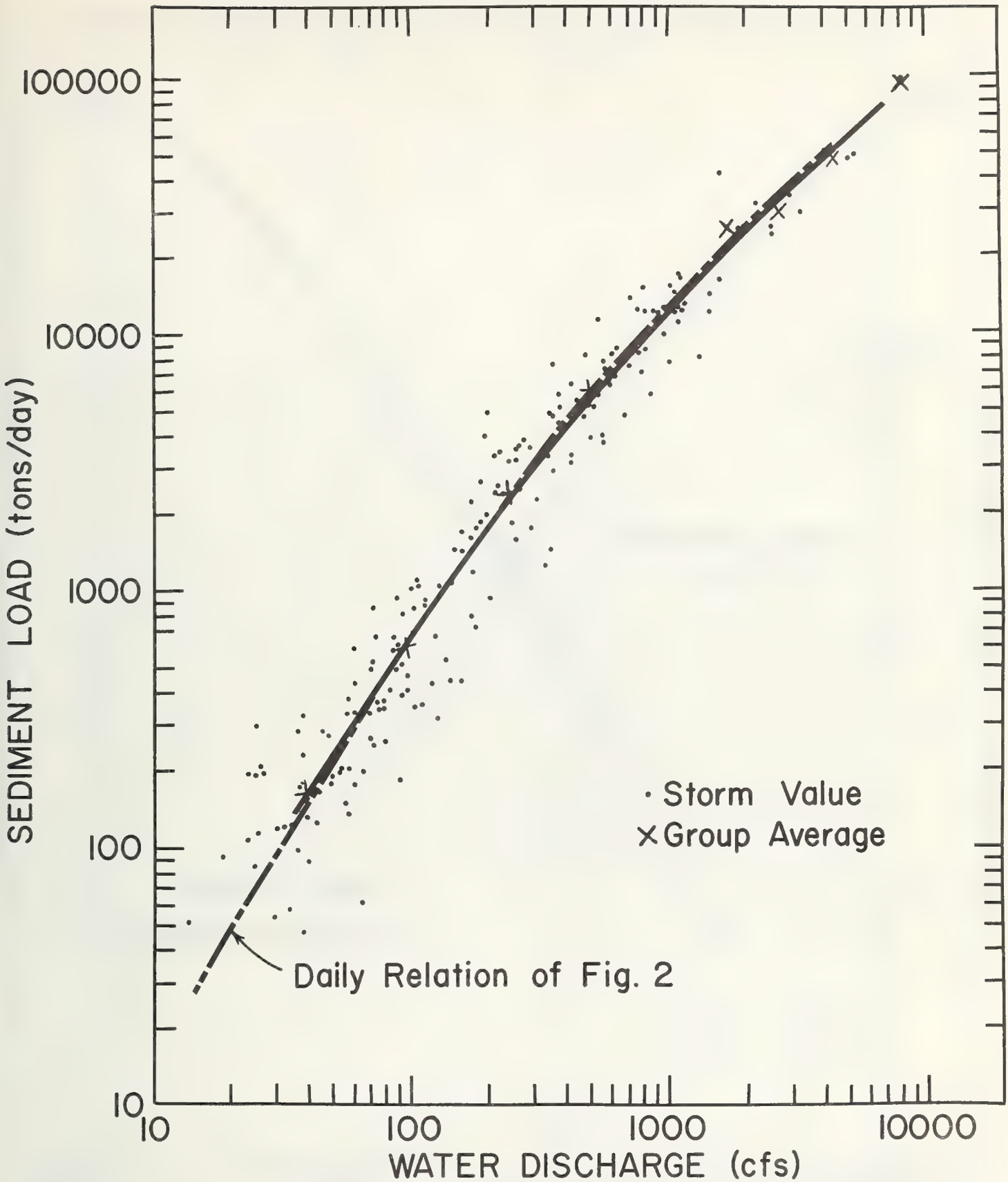

FIGURE 3. — Direct runoff versus sediment discharge, by storm, Pigeon Roost Creek Watershed 34, January 1957–December 1960.

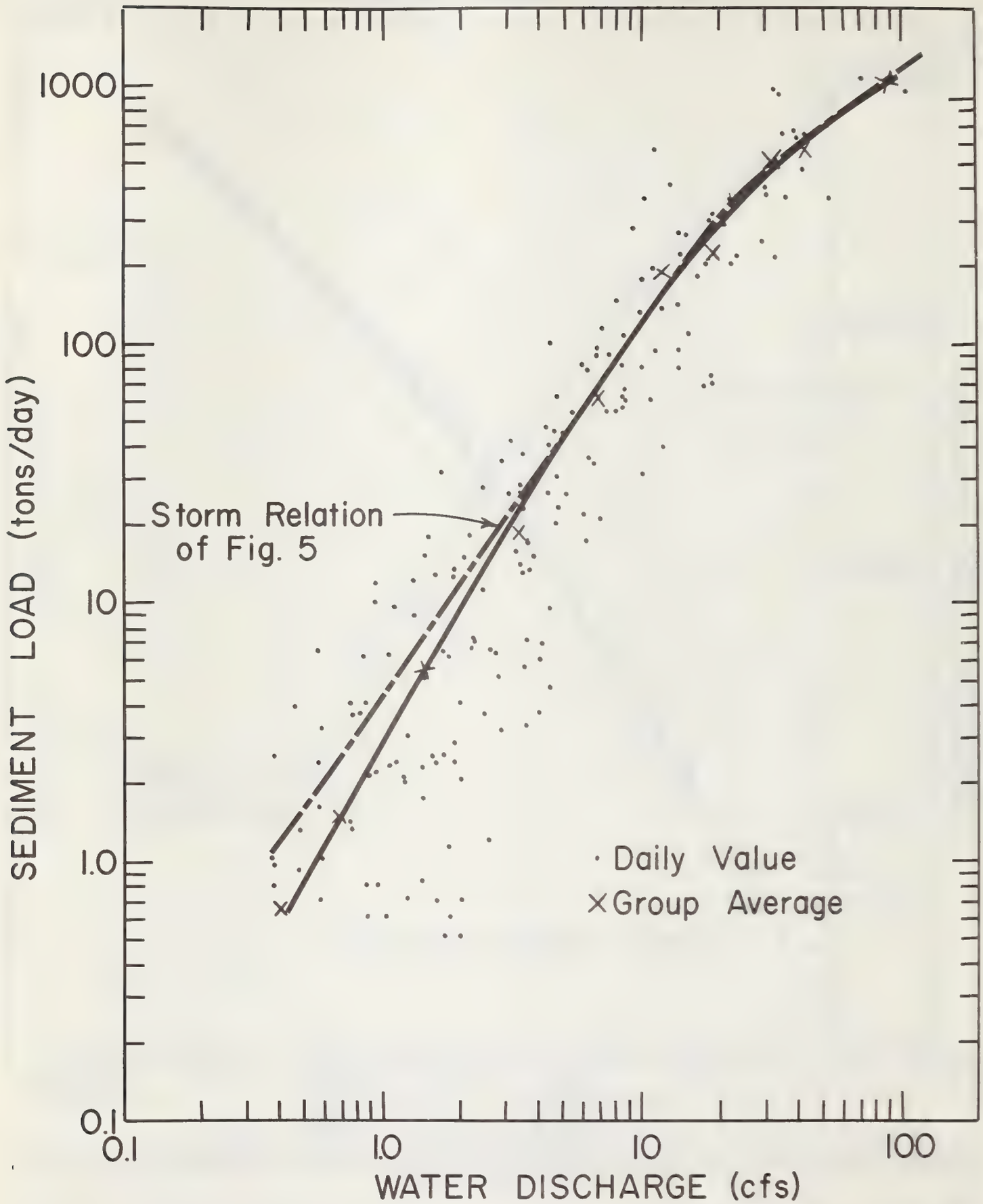

FIGURE 4. — Direct runoff versus sediment discharge, by day, Pigeon Roost Creek Watershed 5, January 1957-December 1960.

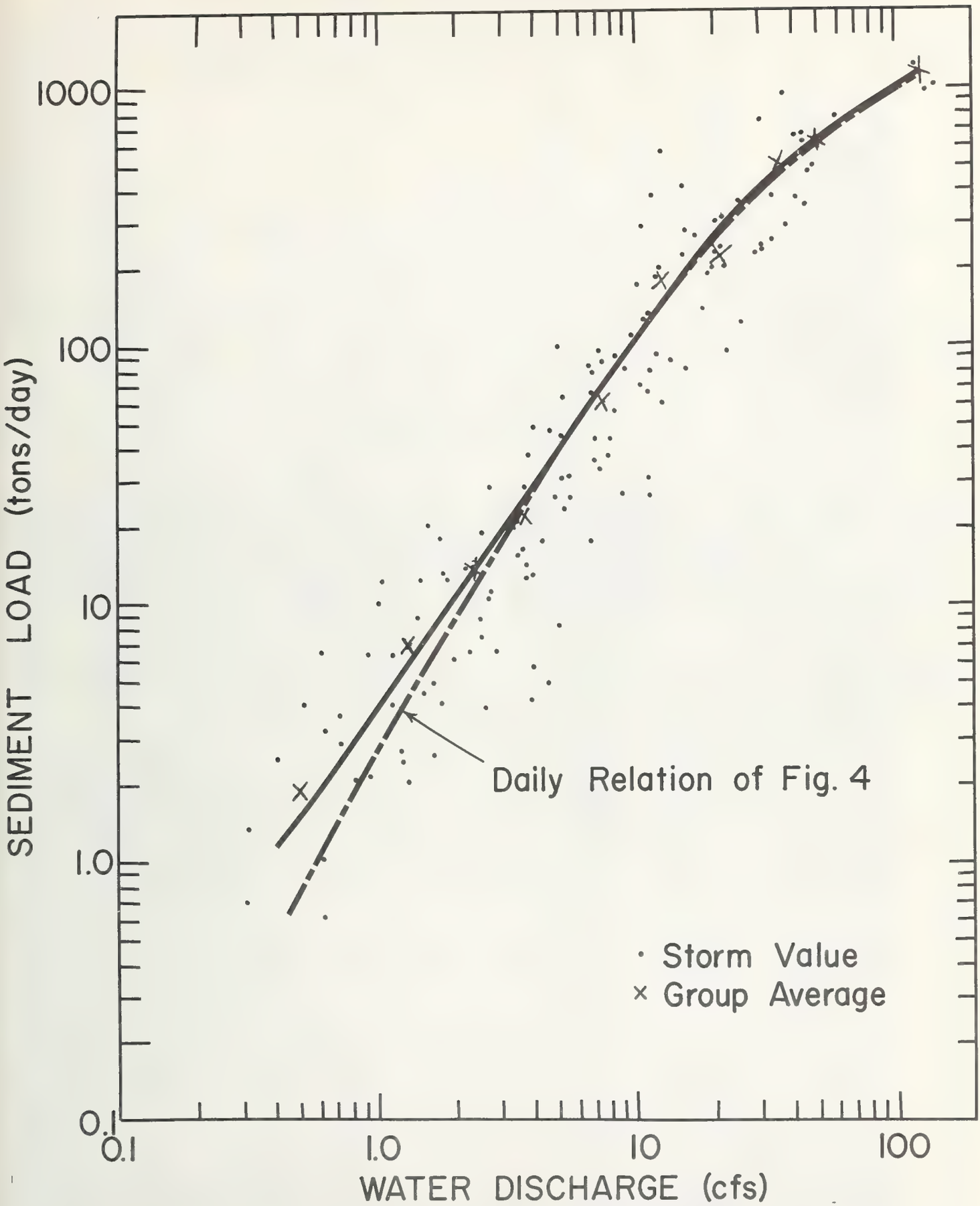

FIGURE 5. — Direct runoff versus sediment discharge, by storm, Pigeon Roost Creek Watershed 5, January 1957–December 1960.

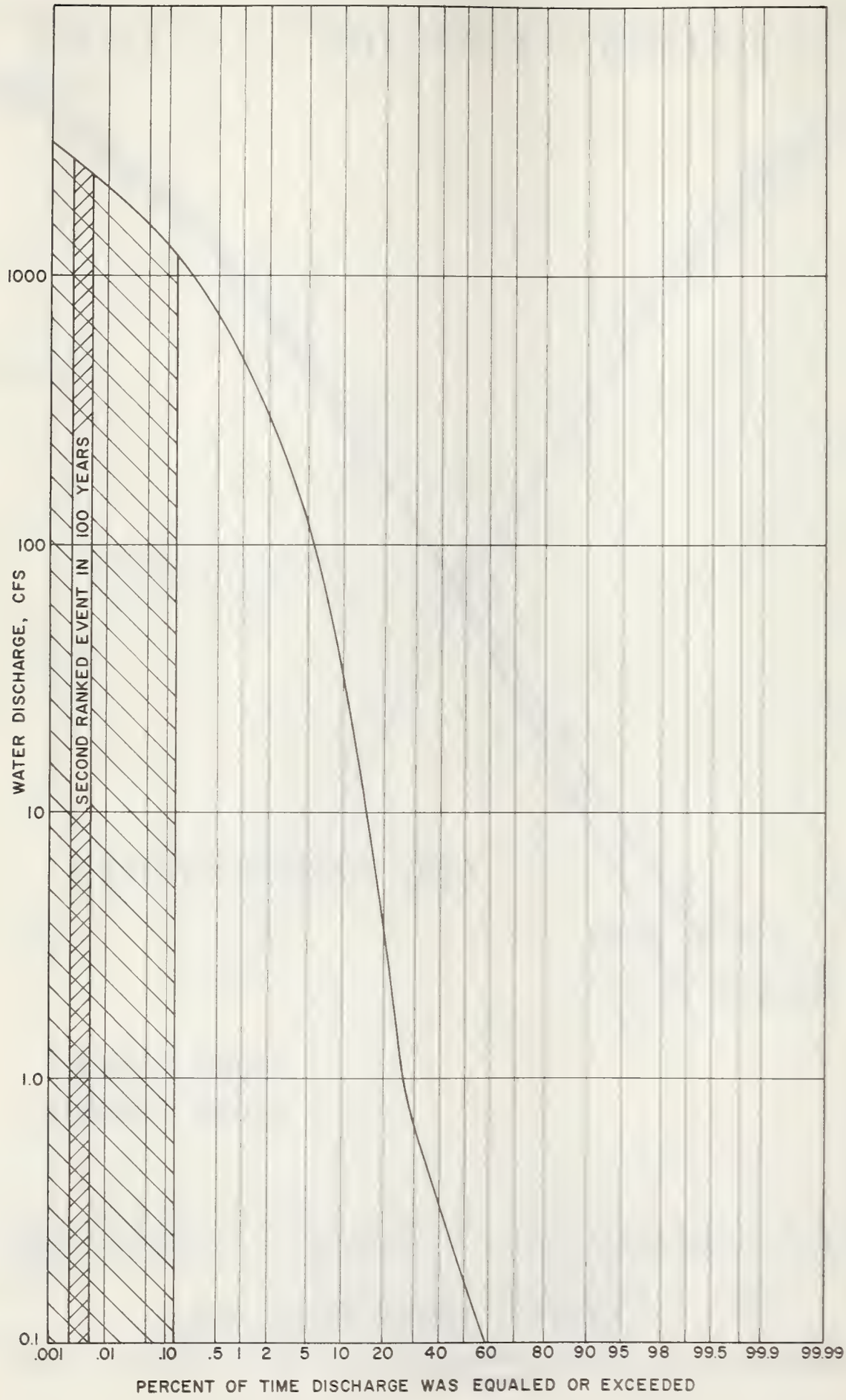

FIGURE 6. — Long term discharge-duration curve representing top-ranked runoff events, Watershed 32, Pigeon Roost Creek Basin, Miss.

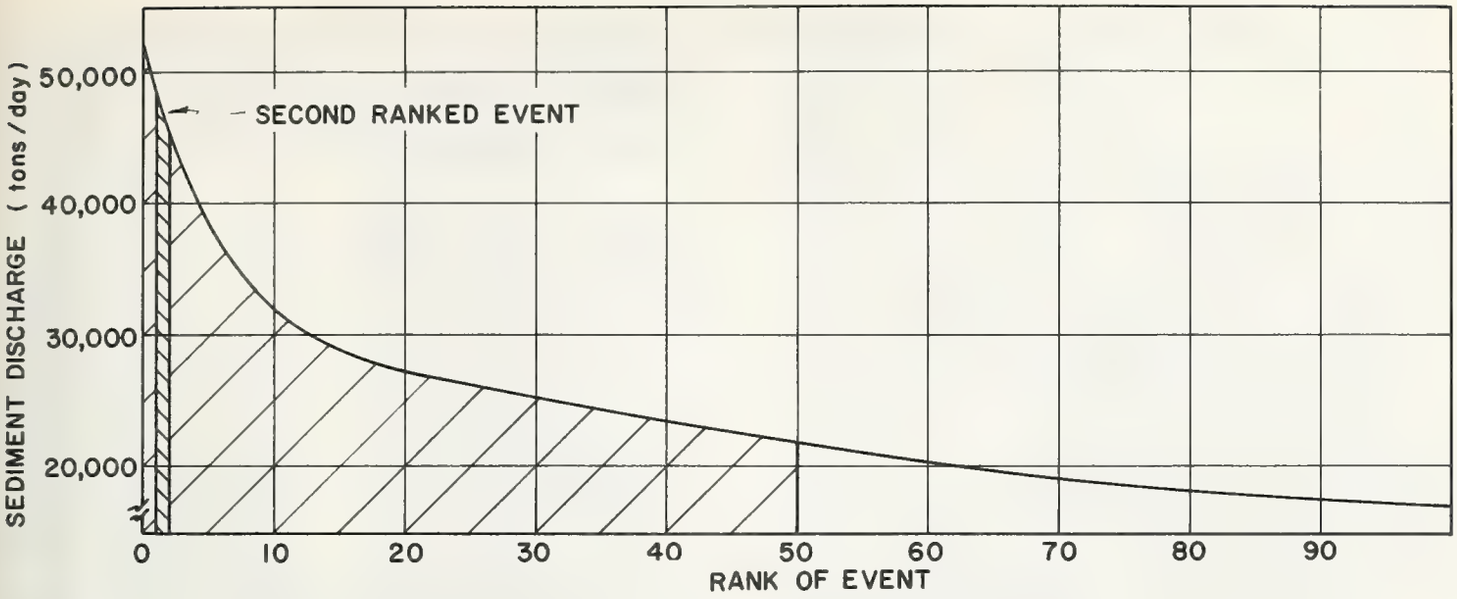

FIGURE 7. — Long term sediment discharge-duration curve representing top-ranked events, watershed 32, Pigeon Roost Creek Basin, Miss.

watershed soil losses resulting from very large storms are proportionally greater in the semiarid Great Plains than in the humid Southeast. Also, the relative sediment contribution of any given large storm tends to decrease with increasing watershed size. These factors are logical, since the average number of runoff events per year are known to vary geographically and with watershed size.

The relative quantities of direct runoff attributable to large storms are substantially less than the relative sediment quantities.

TABLE 1.—*Portion of average annual sediment yield contributed by large, moderate, and small storms*[1]

Watershed	Drainage area	Period of sediment record	Relative contribution by storm type		
			Large	Moderate	Small
	Sq. miles	*Month and year*	*Pct.*	*Pct.*	*Pct.*
Kiowa Creek at Kiowa, Colo.	111	4/56– 9/60	36	16	48
North Clear Creek at Blackhawk, Colo.	55.8	4/52– 6/55	3	5	92
Scantic River at Broad Brook, Conn.	98.4	11/52– 9/56	17	3	80
Davids Creek near Hamlin, Iowa	26	7/52– 9/59	30	11	59
East Fork Hardin Creek near Churdan, Iowa	24	7/52– 9/57	18	10	72
Honey Creek near Russell, Iowa	13.2	7/52– 9/59	17	11	72
Mule Creek near Malvern, Iowa	10.6	7/54– 9/59	39	16	45
Paint Creek at Waterville, Iowa	42.8	11/52– 9/57	18	10	72
Ralston Creek at Iowa City, Iowa	3.01	4/52– 9/59	15	8	77
Tarkio River at Blanchard, Iowa	200	6/34– 6/40	12	7	81
East Limestone Creek near Ionia, Kans.	27.3	4/35– 6/38	32	18	50
Elm Creek near Ionia, Kans.	22.7	4/35– 6/38	44	22	34
West Buffalo Creek near Jewell, Kans.	15.2	4/35– 6/38	33	17	50
Cane Branch near Parkers Lake, Ky.	.67	2/56– 9/58	25	14	61
Helton Branch at Greenwood, Ky.	.85	10/57– 9/58	18	12	70
Plum Creek at Waterford, Ky.	31.9	10/54– 9/58	10	7	83
Watershed 4, Pigeon Roost Creek Basin, Miss.	3.13	1/57–12/60	19	11	70
Watershed 5, Pigeon Roost Creek Basin, Miss.	1.76	1/57–12/60	12	8	80
Watershed 10, Pigeon Roost Creek Basin, Miss.	8.64	1/57–12/60	13	8	79
Watershed 12, Pigeon Roost Creek Basin, Miss.	35.6	1/57–12/60	13	8	79
Watershed 17, Pigeon Roost Creek Basin, Miss.	50.2	1/57–12/60	14	9	77
Watershed 19, Pigeon Roost Creek Basin, Miss.	.38	1/57–12/60	15	9	76
Watershed 24, Pigeon Roost Creek Basin, Miss.	0.80	1/57–12/60	15	9	76
Watershed 30, Pigeon Roost Creek Basin, Miss.	0.18	1/57–12/60	21	12	67
Watershed 32, Pigeon Roost Creek Basin, Miss.	31.13	1/57–12/60	11	7	82
Watershed 34, Pigeon Roost Creek Basin, Miss.	117	1/57–12/60	10	7	83
Watershed 35, Pigeon Roost Creek Basin, Miss.	11.8	1/57–12/60	12	8	80
East Fork Big Creek near Bethany, Mo.	95	4/49– 9/54	24	13	63
West Tarkio Creek near Westboro, Mo.	105	6/34– 6/40	11	7	82
Watershed 3, Hastings, Nebr.	.75	1/57–12/61	29	13	58

TABLE 1.—*Portion of average annual sediment yield contributed by large, moderate, and small storms*[1]

Watershed	Drainage area	Period of sediment record	Relative contribution by storm type		
			Large	Moderate	Small
	Sq. miles	*Month and Year*	*Pct.*	*Pct.*	*Pct.*
Watershed 5, Hastings, Nebr.	.64	1/57–12/61	37	19	44
Brushy Creek near Maywood, Nebr.	130	4/51– 9/58	32	15	53
Dry Creek near Curtis, Nebr.	20	4/51– 9/58	46	15	39
Fox Creek at Curtis, Nebr.	77	4/51– 9/58	42	16	42
Medicine Creek above Harry Strunk Lake, Nebr.	542	4/51– 9/58	37	14	49
Mitchell Creek above Harry Strunk Lake, Nebr.	53	10/51– 9/57	34	15	51
Stoney Brook at Princeton, N.J.	44.5	1/56– 9/58	24	13	63
Kayaderosseros Creek at West Milton, N.Y.	90	2/53– 6/65	13	9	78
East Fork Deep River near High Point, N.C.	13.9	4/34– 6/38	18	10	72
Horse Pen Creek at Battleground, N.C.	15.9	5/34– 6/38	11	7	82
Muddy Creek near Archdale, N.C.	14.2	5/34– 9/40	12	7	81
Uharie River near Trinity, N.C.	11.3	5/34– 8/40	14	9	77
West Fork Deep River at High Point, N.C.	33.0	2/34– 9/40	13	8	79
Little Miami River near Oldtown, Ohio	129	8/52– 9/57	14	8	78
Little Miami River near Selma, Ohio	50.6	9/52– 9/57	11	7	82
Massie Creek at Wilberforce, Ohio	64.3	9/52– 9/58	14	7	79
North Fork Little Miami River near Pitchin, Ohio	29.1	8/52– 8/58	11	7	82
North Fork Massie Creek at Cedarville, Ohio	25.6	7/54– 9/58	13	7	80
South Fork Massie Creek near Cedarville, Ohio	20.2	7/54– 9/58	14	8	78
Todd Fork near Roachester, Ohio	234	9/52– 9/56	16	9	75
Council Creek near Stillwater, Okla.	30.2	10/35– 6/37	14	7	79
Stillwater Creek at Stillwater, Okla.	165	10/35– 6/37	20	12	68
West Fork Brush Creek near Stillwater, Okla.	13.1	10/35– 6/37	23	13	64
Bixler Run near Loysville, Pa.	15	2/54– 9/58	15	9	76
Corey Creek near Mainesburg, Pa.	12.2	5/54– 9/58	24	14	62
Elk Run near Mainesburg, Pa.	10.2	5/54– 9/58	24	12	64
South Tyger River near Reidville, S.C.	106	5/34– 6/38	8	5	87
South Tyger River near Woodruff, S.C.	174	4/34– 6/38	6	4	90
North Tyger River near Moore, S.C.	162	4/34– 6/38	8	5	87
Watershed 1, Riesel, Tex.	.28	1/39–12/47 1/61–12/61	29	16	55
Watershed Y-2, Riesel, Tex.	.21	1/44–12/47 1/61–12/61	28	15	57
Big Elm Creek near Buckholts, Tex.	166	3/34– 9/36	24	12	64
Big Elm Creek near Temple, Tex.	68.6	3/34– 6/36	28	14	58
Deer Creek at Chilton, Tex.	81.8	3/34– 9/36	29	13	58
Elm Fork Trinity River near Muenster, Tex.	46	10/56– 9/58	35	13	52
North Elm Creek near Ben Arnold, Tex.	30.3	10/34– 9/36	25	14	61
Pin Oak Creek near Hubbard, Tex.	17.6	10/56– 9/58	35	14	51
Black Earth Creek near Black Earth, Wis.	45.9	10/54– 9/59	20	10	70
Coon Creek at Coon Valley, Wis.	77.2	4/34– 9/40	25	13	62
Little LaCrosse River near Leon, Wis.	77.1	4/34– 9/40	9	6	85
Mt. Vernon Creek near Mt. Vernon, Wis.	16.1	1/54– 9/59	13	8	79
Yellowstone River near Blanchardsville, Wis.	29.1	8/54– 9/59	16	10	74

[1] With return periods greater than 2 years, less than 2 and greater than 1 year, and less than 1 year, respectively.

GEOLOGY IN SEDIMENT DELIVERY RATIOS

[Paper No. 16]

By SAM B. MANER, *soil conservationist, Engineering and Watershed Planning Unit, Soil Conservation Service*

Introduction

The delivery of erosional material from place of origin to any downstream point is a complex process conditioned by variations in several characteristics of watersheds. These variations are morphometric in origin, primarily topographic in nature, with the principal surficial action agents being hydrologic, hydraulic, and gravitational in character. The complexity of sediment delivery is governed largely by the intensity of the interaction between the variables that shape and control the topographic features of erosional land forms.

In this paper the hydrologic characteristics of drainage basins are considered to be climatic in nature to the extent that rainfall in terms of intensity, frequency, duration, and seasonal distribution and runoff are hydrologically descriptive of an area. Sampling of climatically homogeneous areas eliminates many variations in sediment delivery ratios that could otherwise be attributed to drainage basin variations in hydrologic characteristics.

The hydraulic characteristics of drainage basins, such as channel width, depth, shape, slope, and bed material types, are basically geomorphic properties.

Sediment delivery ratio[1] studies in four physiographic areas indicate characteristics of drainage basins such as channel density, relief–length ratio, and drainage basin size to be significantly related to average annual sediment delivery ratio.

Since the above descriptive terms also are indicative of the topographic features of watersheds, sediment delivery ratio is apparently the function of a group of variables that shape and control the relief features of erosional land forms. The shape and distribution of features of landscapes are conditioned by geologic type and structure, nature and depth of eroding materials, erosion process, and the sequence of topographic development or the stage of the geomorphic cycle.

The relation of sediment delivery ratio variables to the morphological aspects of drainage basin development are discussed in the following pages.

Drainage Basin Size

Drainage basin size is basically a surface reflection of geologic structure and lithology. It has been found to be a significant indicator of sediment delivery ratio in the Blackland Prairies,[2] the Southeastern Piedmont area, (*3*) and the Springfield plain area of Illinois.[3]

In the Blackland Prairies, Maner[2] found drainage basin size to be closely associated with channel density (mi./sq. mi.), watershed relief (ft.), main stem channel length (ft.) and relief–length ratio as well as sediment delivery ratio. These findings indicate basin size to be a "composite–parameter" made up of at least four variables. Thus, in this area, the apparent geomorphic link between drainage basin size and sediment delivery ratio are the morphometric characteristics of drainage basins that control channel density and relief–length ratio (discussed in other sections of this paper) and perhaps other, as yet, unevaluated variables.

One unevaluated aspect of the sediment delivery–drainage basin size relation is the hydrologic-hydraulic influences. In the field of hydrology and stream hydraulics it is generally known that drainage basin size is related to discharge volume per unit area, time of concentration, etc. However, little is known in regard to how these factors may influence the sediment delivery ratios of drainage basins. Wolman and Miller (*7*) have pointed out the probable nature of some of these relations in their study of magnitude and frequency of forces in geomorphic processes. They show drainage basin size to have a somewhat unique relation to the frequency characteristics of sediment transport. For example, for drainage basins larger than 8,000 to 9,000 square miles, less than 50 percent of the average annual runoff is produced by storms having a frequency of once each year. In contrast, storms of similar frequency account for more than 70 percent of the average annual runoff from drainage basins ranging in size from 0.1 to 3.0 square miles. This characteristic discharge, frequency, annual runoff relationship, coupled with the fact of higher discharge per unit area produced by smaller areas, would indicate higher sediment delivery ratios to be associated with smaller drainage basins.

Perhaps another significant aspect of the drainage basin size–sediment delivery ratio could be explained within the framework of the concept that geomorphic cycle stage changes not only with time but with distance from the drainage basin mouth. On this basis the intensity of topographic development in terms of relief, channel density, valley side slopes, stream gradients, etc. moves upstream, with the youthful and early maturity part of the geomorphic cycle followed by late maturity and old age. As one moves upstream, the drainage basin area decreases and the topographic factors that promote sediment delivery become more intensified, resulting in higher sediment delivery ratios. Strahler (*6*), in an analysis of drainage basin topography, developed the hypsometric integral as a measure of geomorphic cycle stage, and this integral may be of value in relating drainage basin size to sediment delivery ratio.

Channel Density

The apparent link between channel density and sediment delivery ratios, especially where the major source of sediment is from sheet erosion, is the distance over which sediment laden, relatively shallow, low velocity runoff must travel before concentrating in permanent drainage channels. Any increase in channel density reduces the possibilities of deposition of erosional material between the place of entrainment and the downslope point where channelized flow begins. Channel density is, therefore, an index of the efficiency of a drainage network in collecting runoff delivering erosional material to any downstream point.

The degree of channelization for a given size drainage basin on structurally similar geologic areas, in a given climatic zone, is primarily dependent upon drainage basin lithology. Lithology, in turn, is a major factor in

[1] The ratio of sediment yield to gross erosion.

[2] MANER, S. B. FACTORS INFLUENCING SEDIMENT DELIVERY RATIOS IN THE BLACKLAND PRAIRIES LAND RESOURCE AREA. U.S. Dept. Agr. Soil Conserv. Serv. [Filed at SCS Fort Worth, Tex., office.]

[3] MANER, S. B., and GEIGER, A. F. [Unpublished study from files of U.S. Department of Agriculture, Soil Conservation Service, at Forth Worth, Tex.]

determining the resistance of surficial materials to erosion, due to its relationship to surface and subsurface permeability. In general, resistance to erosion is less in rocks of low permeability. Hence, drainage basins on shales and similar fine-grained sedimentary rocks commonly have high channel density values and consequently higher sediment delivery ratio values. Drainage basins on coarse-grained clastic rocks, in contrast, tend to have fewer miles of channels per square mile of drainage area and a corresponding lower sediment delivery ratio.

In the Blackland Prairies, a relatively homogeneous area lithologically, Maner [4] found channel density to be closely associated with drainage basin size. The results of this study indicate the lithology or textural characteristics of the surface mantle to be a prime factor in intensity of channel development if basin area remains approximately constant. Any increase or decrease in basin area results in a decrease or an increase in channel density, respectively. This association of drainage basin size with channel density is believed to be due to the increase in relief per unit of area associated with reduction in drainage basin area and the runoff intensity-frequency character of decreasing drainage basin area.

Some other geomorphic characteristics of drainage basins that have been found to be associated with channel density or drainage density include relief ratio (*6*), relief-length ratio,[4] stream gradients (*5*),[4] and valley side slopes (*2*, *6*).

Relief-Length Ratio[5]

Drainage basin relief and length expressed as relief-length ratio have been found to be significantly related to sediment delivery ratio in four widely separated, morphologically different physiographic areas (*1*, *3*).[6] Schumm (*4*) found a close association between soil loss and relief ratio in a study of several "stockpond"-type reservoirs in Utah, Arizona, and New Mexico.

Quantitatively, relief-length ratio is an index of drainage basin concavity as well as of the cross sectional slope characteristics of a drainage basin. Morphologically, it is an excellent indicator of several topographic characteristics of drainage basins. For example, Schumm (*4*) found relief ratio (essentially the same as relief-length ratio) to be closely correlated with stream gradients, valley side slopes, drainage density, and basin shape in a study of drainage system evolution and slope development in a badlands area near Perth Amboy, N.J. Maner [7] found relief-length ratio to be significantly related to drainage basin size, channel slope, and channel density in the Blackland Prairies of Texas.

Results obtained in the Red Hills physiographic area indicate relief-length ratio may be an excellent parameter for estimating sediment delivery ratio for a group of drainage basins that differ considerably in structure, lithology, size, and other geomorphic characteristics.

Conclusions

The link between drainage basin geology and sediment delivery ratio is the drainage-network elements. The initial scale, shape, and distribution of these elements are a function of geologic structure and lithology. The current shape, scale, and distribution of these elements are conditioned by the kinds of geomorphic processes active in a given area and the length of time the processes have been active. The fact that interrelation between geomorphic characteristics of drainage basins does exist indicates the current stage of the geomorphic cycle to be a prime factor in determining which of the drainage basin characteristics are best related, quantitatively, to sediment delivery ratio.

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[4] See footnote 2.

[5] Watershed relief, in feet (defined as difference in elevation between average elevation of the watershed divide at headwaters of main stem drainage and elevation of streambed at damsite), divided by maximum watershed length, in feet, measured approximately parallel to the main stem drainage from damsite to watershed divide.

[6] See footnotes 2 and 3.

[7] See footnote 2.

GULLY CONTROL METHODS IN IOWA

[Paper No. 17]

By PAUL JACOBSON, *state conservation engineer, U.S. Soil Conservation Service*

Gully problems in Iowa vary significantly as one goes from the deep loess area in western Iowa to the area of shallow loess over till in eastern Iowa.

In the Ida-Monona-Hamburg area of western Iowa gullies are characterized by deep entrenchment with nearly vertical banks. Gully depths in this area will range to 100 feet, with a great number in the range of 30- to 40-foot depths. In the Marshall area gullies are similar to the Ida-Monona area to which it is adjacent, but they are not as deep. Generally they are about 15 to 25 feet deep, with a few ranging to as much as 30 feet in depth.

In the shallow loess over heavy till area of southeastern Iowa, gullies take on different characteristics. They are of much less depth and the shape changes from vertical banks to sloping banks. The slope of the banks generally is about 2 to 1, and they tend to become vegetated over a period of years. This type of gully exists all through southern and eastern Iowa with the exception of the Fayette area. In the Fayette area, gullies are similar to the deep loess area of western Iowa, except that depth is limited by interception with limestone in the bottom of the gully.

Gully problems in the level soils of north-central Iowa are very limited, with only an occasional gully along the major streams requiring treatment.

Development of Gullies

The rate of gully advancement, as indicated by Thompson,[1] varies more, owing to the size of drainage area than to most other factors. Rates of gully advance within soils areas in Iowa probably have not varied significantly. However, the character of the gully to some extent determines the degree to which people are willing to live with gullies. In the loess area deep gullies are associated with a corresponding greater width as time progresses. This greater width causes much higher land destruction by a gully of comparable drainage area. Also, the great depth of the gullies and the vertical banks in this area have made it difficult to provide access to cropland areas already dissected by deep gullies.

The corresponding demand for gully control by local people has been much greater in deep loess areas.

Present Gully Control Methods for Deep Loess Areas

Present gully control methods for deep loess areas are similar to methods used in eastern Iowa in that the main emphasis is to provide for water disposal. Generally, structures have been located so they would submerge overfalls, or they have been located so a grassed waterway could be built to provide protection for the overfall. Due to the depth of gullies, the height of head necessary for control has been rather large. The type of structure used has consisted mainly of pipe drop inlets that store part of the runoff and provide a slow release rate through the conduit. In order to be assured that the temporary storage pool is available during the life of the structure, sediment storage capacity must be provided. Estimated sediment deposits expected during the future 50-year period are based upon the land use and treatment of the drainage area. Another structure used to a lesser extent in this area is the concrete chute. It also is adapted to control the high heads required by these deep gullies; however, the cost of this type of structure often exceeds that of a drop inlet.

Are Present Gully Control Methods Doing the Job?

When consideration is given to the major problems of western Iowa, it might be well to analyze present goals and objectives of the gully control program. One of the major problems in obtaining maximum yields in this area is the shortage of moisture. Thus, one goal should be to store water on the land where it falls so it can be used for crop production. The soils of this area are well suited to making this goal possible. They have high infiltration rates and large water-holding capacity.[2]

Possible Improvement of Western Iowa Gully Control Methods

All gully control methods, as has been done in the past, should be coordinated with the necessary land treatment needed. In the deep loess area, sheet erosion has been extremely high under the cropping system and the land treatment practices being followed. Over a 50-year period soil losses of as much as 5 or 6 [3] inches occur in the watershed.

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[1] THOMPSON, J. R. QUANTITATIVE EFFECT OF WATERSHED VARIABLES ON THE RATE OF HEAD ADVANCEMENT. Amer. Soc. Agr. Engin. Meeting, Paper 62-713. Chicago. 1962.

[2] BROWNING, G. M. EFFECTIVE UTILIZATION OF WATER IN HUMID AND SUBHUMID AGRICULTURAL AREAS. Natl. Water Res. Symposium Proc. March 28 to 30, 1961. Washington, D.C.

[3] Unpublished material from studies on Mule Creek, Nepper, and Theobold Watersheds, Iowa.

This suggests that any gully control method now being practiced will bench the valley areas. It also suggests that if gully control is to be feasible, sheet erosion on the hillside will need to be controlled. This control probably can be achieved by two methods: (1) Retiring most of the hill ground to permanent grass; (2) bench terracing the hillsides.[4]

In all probability, based on the present relatively intense use of this land and the high productivity possible on these deep loess soils, the first alternative will not be accepted by farmers.

Level terraces, however, have been fairly well accepted in western Iowa. They provide a means whereby much of the runoff can be held until it soaks into the soil.[5] In dry years the yields of corn in the terraced channel have been recorded at 105 bushels to the acre vs. 41 in the sloping area between terraces. This suggests that terraces should received increased acceptance.

Since gully development is dependent on drainage area and the accompanying runoff, it follows that if drainage area at the overfall is eliminated by level terraces, gully advancement for the main part can be controlled by completely terracing the area. This leaves three major purposes for gully control structures.

(1) To furnish crossings on a farm to get to fields that have been isolated by the deep vertical-sided gullies.

(2) To bench valleys, making level areas above the structure suitable for farming and incidentally reclaiming large areas of land that are presently voided by the large gullies.[6]

(3) To furnish water areas suitable for recreation and wildlife.

On item 2 there could be considerable development in western Iowa. If structures are used primarily to bench land and to develop suitable conditions for maximized farming of valley areas, structure locations will change from those presently being used and recommended. The structures probably will be moved downstream to a point where the maximum area of farmland would be provided in the valley after the benching process has been completed.

[4] JACOBSON, PAUL. A NEW METHOD FOR BENCH TERRACING STEEP SLOPES. Amer. Soc. Agr. Engin. Paper 62-716A. Chicago. 1962.

[5] Annual Reports, Bluffs Fruit Farm, Iowa State University.

[6] MESSINES, JEAN. FOREST REHABILITATION AND SOIL CONSERVATION IN CHINA, 1962. Translation to English, Unasylva 2 (No. 6), p. 103.

[7] HENDRICKS, E. L. PHILOSOPHY OF WATER DEVELOPMENT. Natl. Water Res. Symposium Proc., Mar. 28 to 30, 1961. Washington, D.C.

Jones Creek Is Example of Valley Benching

About 20 years ago the Jones Creek Watershed was planned and installed in Monona County in western Iowa. The watershed is in the Hamburg area where upland slopes, which are mainly over 20 percent, have remained in permanent native vegetation. The valleys were dissected by deep gullies that had voided large parts of the farmland area. Structures were located at the lower end of gully junctions, so they would develop benches in the valley with sufficient area so they would be suitable for farming. The structures were designed as full-flow concrete chutes. Provisions were made initially so these structures could be raised when sediment had deposited to the lip of the structure. Benches have developed above these structures to or above the original lip of the chute. The structures have not been raised, but nevertheless they have demonstrated that valley benching is desirable and feasible. It can be observed that after structures are filled with sediment and filling has progressed above the structure lip, new gullies at a higher level will develop in the sediment plain above the structure. This will be true, unless structure heights are raised at a rate about equal to the deposition of sediment above the inlet of the structure. This process will go on until benches are at such a grade that no further soil movement will occur.

The Future of Gully Control

This discussion indicates that gully control methods presently being used in western Iowa need some reorientation of objectives and goals. As more and more level terraces are installed on hillsides, drainage areas of overfalls and the corresponding runoff will be reduced.

As terraces are installed, the level area in the valley remaining below the terraced area becomes more like the Jones Creek Valley and valley benching on many areas becomes possible. This is probably looking at the ultimate plan for the development of the deep loess area of western Iowa, but it does follow the pattern of proposed development of similar deep loess areas throughout the world. As more and more water is stored on the hillsides and in the valleys and as more and more areas approach being level, surface runoff will be reduced. This will probably be accompanied by rising underground water tables and it is probably that tile drainage will be required in the valleys. Flood control provided by this type of program would be by storing water in the soil itself, which was suggested by Hendricks.[7] The feasibility of this is also pointed out by 10 years of runoff studies on Jones Creek by Iowa State University during the cropping season where the maximum annual

runoff has been 0.8 inch.[8] Thus, it would seem that gully control in this deep loess area should be to provide one of the three objectives listed previously.

New Studies

New studies are being initiated in western Iowa to verify the practicability of controlling gullies by complete terracing of the hillside. Runoff studies are being made in the Macedonia watershed on which a large part of the 400 acres of upland watershed has been terraced. Also, land has been purchased for 4 small watersheds of 90 to 160 acres to study runoff and gully head advance where the area is in grass, completely terraced, or in a row crop, without terracing.

Gully Control in Shallow Loess Over Till Areas

As we move into the shallow loess over till area in Iowa, our gully treatment program should have a different objective. In this area we do not have deep soils that are capable of storing large quantities of water temporarily which can later be removed by crops or tile. Here it will be necessary for the gully control structure to be one of the steps to provide a stable channel for removal of excess water. The land treatment program will consist of a cropping system and the necessary terraces to control erosion and reduce sediment to tolerable limits. Depending upon the intensity of cropping, the usual treatment may consist of a terrace or two at the top of the hill in the shallow loess soil. The terraces as used will lead the water to a hillside channel or waterway that can be stabilized. The remaining water disposal system will depend for the most part on developing a grassed waterway to remove runoff water at a velocity that will be safe for grass. A structure in this case will be needed to furnish transition from the broad waterway to the narrower downstream channel. The downstream channel is normally stable, because of its reduced grade. In this area, the gully control structure will have two of the reasons listed for western Iowa and a third objective as follows: (1) Water storage for recreation; (2) a structure to furnish a crossing to areas that have been made inaccessible by gully dissection; and (3) a transition from a broad waterway cross section to the narrower stream channel cross section.

If recreation is desirable and large areas of highly productive land will not be covered by water, the gully control can be accomplished by a pipe drop-inlet type of structure. Sediment storage will be provided for the estimated life of the structure, with enough additional volume provided for gully control and recreational purposes. This type of structure furnishes an ideal transition from the broad upstream waterway to a narrower downstream, entrenched watercourse. In addition to furnishing a transition, this type of structure provides for reduced downstream flow. To some minor extent, this will reduce downstream floods and to a greater extent it will provide for more downstream stabilization than where normal quantities of runoff must be provided.

In other cases where it is merely necessary to provide a transition from a broad waterway to a narrower downstream channel, a toe wall, a drop spillway, or a drop structure on a road culvert will provide the necessary transition. The depth of gully at the transition will generally determine whether a toe wall or a drop spillway is used. Where the channel at the transition is less than 4 feet deep, a toe wall is used. For greater depths a drop spillway is used.

Economics of Control Measures

Where gully control measures are proposed, the benefits derived should exceed the cost of the control measures. In watershed work a study is made of the reduction of damage by gully encroachment to upland areas by the installation of the control. Also reduction of downstream damage due to flooding, or covering of cropland by infertile overwash, or filling of drainage channels is enumerated, and the damage computed.

Similar studies are sometimes conducted where gully control measures are installed on a cost-sharing basis. Or, if the farmer is willing to invest money, this indicates the enomomic feasibility of the control measures.

Summary

1. The necessary land treatment program always should be applied to the land as part of the gully control program.

2. It is probable that as means are found for storing more water in the soil for use by crops that the number of structures needed for gully control can be reduced.

3. Gully control structures should be planned for multiple use, such as benching the land, furnishing recreation, providing part of a water disposal system, and furnishing road crossings across the gullies.

4. Many of the ideas presented in this paper are based on observations or limited research material. More research is needed.

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SEDIMENT REDUCTION THROUGH WATERSHED REHABILITATION

[Paper No. 18]

By EDWARD L. NOBLE, *forester, Intermountain Region, Forest Service, Ogden, Utah*

History is replete with stories recounting the failures of man to recognize, control, and conquer the devastating effects of sediments from steep mountainous lands. Learned men have documented the reasons for the tragic downfall of highly developed civilizations in Mesopotamia, Israel, Egypt, and elsewhere. Most agree that it was not conquest of the land by an invader nor the loss of fertile fields that depopulated the land, but the relentless encroachment of silt into the canals and rivers that forced the people to move elsewhere or starve.

We may now say 7,000 years later that such an occurrence could not happen again; that our engineering skills and general knowledge preclude such disasters that befell the ancient civilizations (*8*). In part, such an assumption is true, for we have expended billions of dollars to construct dams, levees, canals, and intricate irrigation systems. With such developments, we have assured ourselves of a continuing expansion of irrigated agriculture, industry, and attendant municipalities.

There is no question of our awareness of the critical water shortage problem that faces America today. Such awareness was well documented by the voluminous testimony before the Senate Select Committee on National Water Resources. Nor is there a question of the need for continued development of entire river basins to obtain water for the growing industrial and population expansion which is so imminent. Such recognition is well reflected by the huge annual appropriation of funds for water development and flood control purposes by both Federal and State governments.

With such concrete recognition of the water problems and the vast programs now underway and planned, you may reflect that we have or will have the situation under control and that the continued construction of dams and other structural projects will provide the utopia which we are presumably seeking.

No argument prevails that such programs are vital and essential to insure an expanding economy and continuing prosperity. However, there is a question of their continued success if we ignore the need for onsite uses of water and fail to recognize the hydrologic conditions on the watersheds themselves.

The principle of regulatory stream behavior and maintaining soil stability through land management is not new. It was advanced as a guiding principle over 50 years ago by the late geologist, Thomas C. Chamberlin, who stated, "The key lies in due control of the water which falls on each acre . . . The highest crop value will usually be secured where the soil is made to absorb as much rainfall and snowfall as practical . . . This gives a minimum of wash to foul the streams, to spread over the bottom-lands, to clog the reservoirs, to waste the water power, and to bar up the navigable rivers."

Since the time of Dr. Chamberlin's keen observation, we have learned much regarding the hydrology of watershed lands. The depths of the soil have been probed, and the interrelation between soil cover, precipitation patterns, and water disposal have been critically examined. By so doing, the research workers have established the fact that a very fundamental relation exists between land condition and hydrologic behavior. We have also learned through experience and the practical application of research results that upstream engineering and improvement of watershed conditions can greatly reduce land erosion and the resultant damages of sedimentation.

Recognizing, then, the fundamental relation between land condition and watershed behavior, we are able to set as our objective the maintenance of both the productive and the hydrologic or water-regulating functions of the land. By achieving this objective, we are able to obtain the greatest quantity of forage and fiber from the land and simultaneously make certain that water is yielded with the greatest possible regularity and with the least possible load of sediment.

In developing a program of sediment reduction through watershed rehabilitation it is necessary to ascertain fundamental facts such as (1) the geologic norm, (2) type of flooding, (3) watershed protection requirements, and (4) adaptability of the site for treatment.

Geologic Norm

Essential to the formulation of any flood and erosion control program through watershed rehabilitation is an understanding of the relation between current flooding and erosion to the geologic norm (*1*, *3*). This is necessary as watershed rehabilitation measures are primarily aimed at controlling water runoff and erosion from those areas on which accelerated erosion is occurring.

Watersheds with their soil and plant mantle, topography, and streamflow character have been inherited from the geologic past. Their

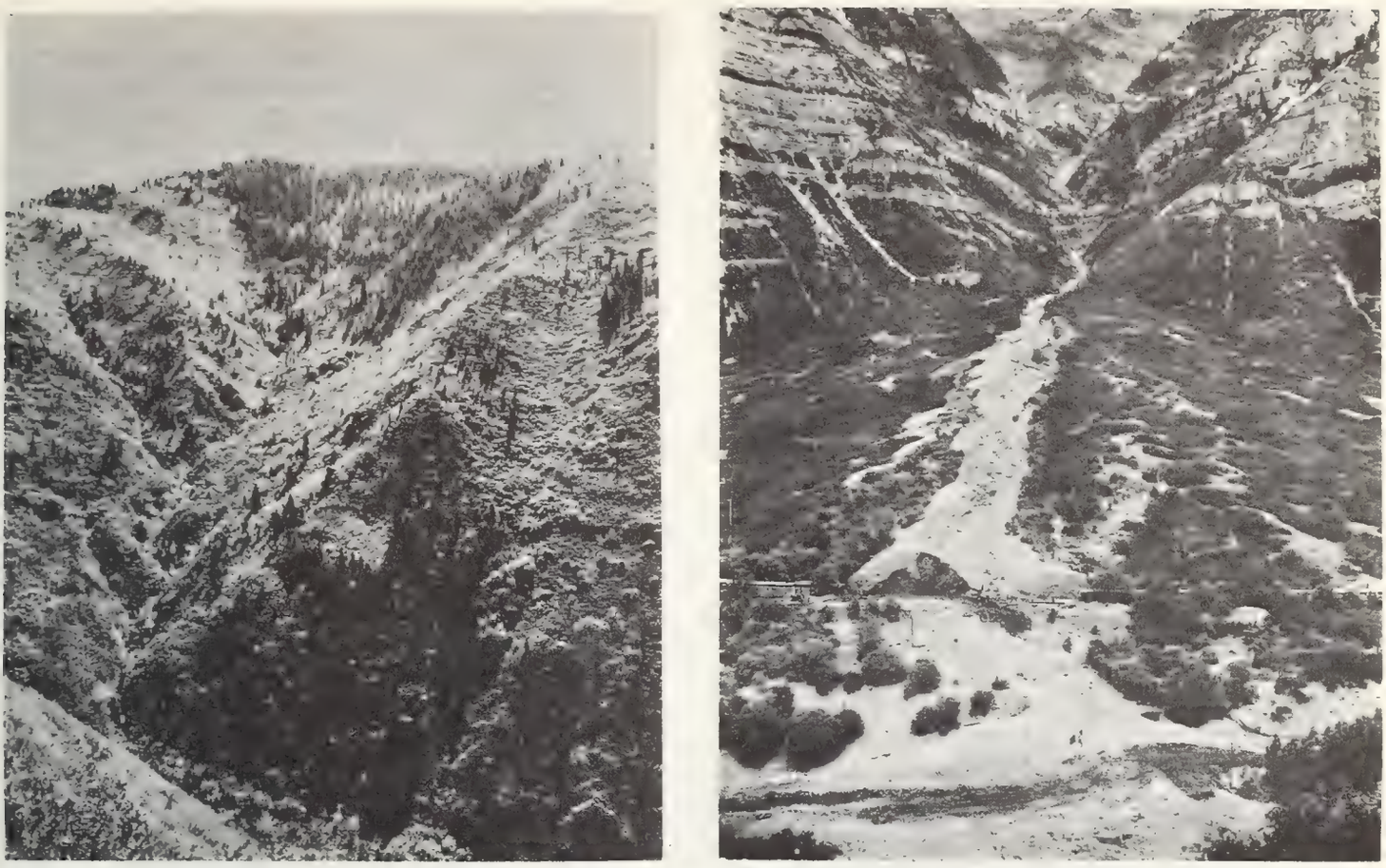

FIGURE 1. — Left, Morris Creek Watershed, Utah, north-facing basin whose average slope is 48 percent with extreme gradients of more than 80 percent. Precipitation averages 30 inches annually, which is completely infiltrated. Dense vegetation provides protection against erosion. Sediment production from this watershed is only 0.0025 acre-foot per square mile per year and represents a low geologic norm (*6*). Right, Lost Creek Watershed, Utah; 85 percent of area comprised of steep barren slopes. Active erosion and periodic flooding are characteristic of this basin, giving it a high geologic norm. Small islands of soil protected by vegetation show no evidence of overland flow and erosion. Erosion removes soil material as rapidly as it forms from unvegetated slopes (*6*).

streams exhibit great natural variation in erosion and flood behavior. Some streams are usually clear and flow with a relatively constant volume, but the regimen of other streams is marked by great variance in volume and time of flow and vast differences in sediment content. Each stream is the resultant of such normal factors and forces as climate, topography, geology, and the plant and soil mantle. We know, for example, that erosion is proceeding so slowly in some areas that soil is being formed and accumulated more rapidly than it is being removed. Streams from such areas carry only negligible loads of sediment. We know, too, that in other areas climatic and geologic conditions limit soil formation, plant growth, and fixing of the land surface. From these drainages runoff has always been rapid and erosion pronounced, giving rise to muddy and highly fluctuating streams. Moreover, we know that between these extremes are all gradations of watershed and sedimentation rates (*6*). Variation in sediment production in relation to watershed conditions are shown in figure 1.

From some knowledge of the geologic norm, a study of the condition of the watershed, and a determination of the history of runoff from a flooding stream, we can determine whether erosion is accelerated or normal. If accelerated, an opportunity for rehabilitation exists.

Type of Flooding

Flooding from high mountain watersheds usually occurs as wet-mantle floods, due to rapid snowmelt runoff, or as dry-mantle floods resulting from high intensity summer rainstorms.

Although watershed rehabilitation measures have proved valuable in preventing serious damage from both types, they are primarily used to control surface runoff of water and erosion from floods classed as dry mantle. Accordingly, a knowledge of the characteristics of dry- and wet-mantle floods is necessary in order to design appropriate treatment measures.

Some characteristics of dry-mantle and wet-mantle floods (7)

Factor	*Dry-mantle flood*	*Wet-mantle flood*
Soil mantle-condition.	Dry—high water storage capacity.	Wet—storage capacity exhausted.
Precipitation	Short, intense rainfall.	Prolonged rainfall or snowmelt, or both.
Storm area	Usually small—may be only 5 to 10 percent of flooding drainage.	Relatively large—usually all of flooding drainage.
Volume of water	Relatively small—may be only a few acre-feet.	Relatively large—thousands of acre-feet.
Manner of flow to stream channels.	Over surface	Mainly seepage or "bleeding" of saturated soil mantle.
Sediment carried	High—may reach 60 percent of volume.	Low—in relation to water volume.

Watershed Protection Requirements

Since 1920, much research work has been done on high mountain watershed lands to determine the relation between ground cover (including plants and litter), precipitation, surface runoff, and erosion.

After floods, field investigations have been made to determine flood-source areas, their condition, and their specific contribution to runoff and sedimentation. From these many analyses it has been determined that heavy flooding has occurred when the hydrologic balance on as little as 2 to 10 percent of the watershed has become deteriorated. It has also been noted that such deterioration is primarily due to a reduction of cover to the extent that each area of land was unable to receive and dispose of water through the process of infiltration.

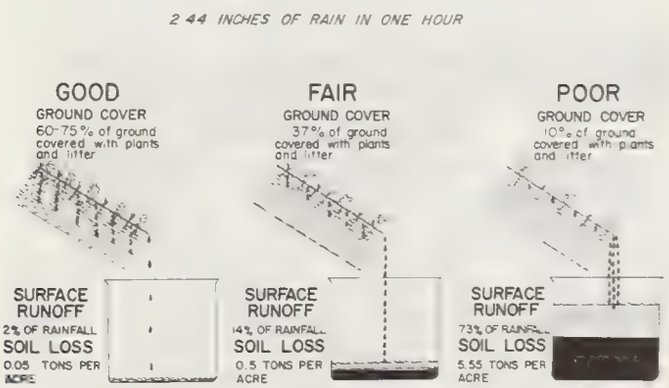

FIGURE 2.—The effect of watershed condition on rainstorm runoff and erosion, Subalpine Range, Ephraim Watershed, Utah.

Research efforts have covered a wide variety of soil conditions and vegetal types. Results indicate that in the intermountain area, a minimum of 60 to 70 percent ground cover is needed to control effectively surface runoff of water and erosion occasioned by torrential summer rainstorms. The same studies have also indicated that when ground cover has been reduced below 60 to 70 percent, overland flow and soil losses increased at an extremely rapid rate. The relation of ground cover to surface runoff and eroded soil for different soil-vegetation complexes is shown in figures 2, 3, and 4.

Determining Type of Watershed Rehabilitation

After an analysis of a flood source area to determine the type of erosion, type of flooding, and ground cover condition, it is then possible to plan the watershed treatment required. In most cases the immediate objective is to restore the watershed site to a condition where precipitation is received and disposed of without the occurrence of oversurface flows of water (*4*, *9*). Based on several factors, watershed rehabilitation efforts fall into the following general categories:

1. Where sufficient native plants are available to furnish a seed supply, adequately fertile soil is present, ground cover density exceeds 40 percent, and a definite erosion pattern of rills and gullies is not evident, restoration of the hydrologic balance can be obtained by means of natural revegetation. This process may be slow; in most instances, very conservative management practices, such as rest-rotation grazing and light use by livestock, are required.

2. Where natural revegetation cannot be expected because of lack of seed, but where soil is productive and slopes do not exceed 30 percent gradient, artificial revegetation, using approved range seeding methods, are necessary to obtain needed plant cover. This method of rehabilitation requires careful selection of plant species, intensive farming methods, and complete protection from use after treatment until the vegetation is well established. Often it is desirable to supplement this treatment with the installation of a series of contour furrows to prevent surface runoff.

3. Where both plants and soil fertility have been destroyed so completely that a protective cover cannot be established by categories 1 or 2 above, the soil must be stabilized and surface runoff controlled by mechanical devices until the plant cover can be restored. To achieve this, a system of contour trenches and gully plugs is constructed and areas of soil disturbed by construction activities are seeded to grasses or planted to trees or shrubs (fig. 5).

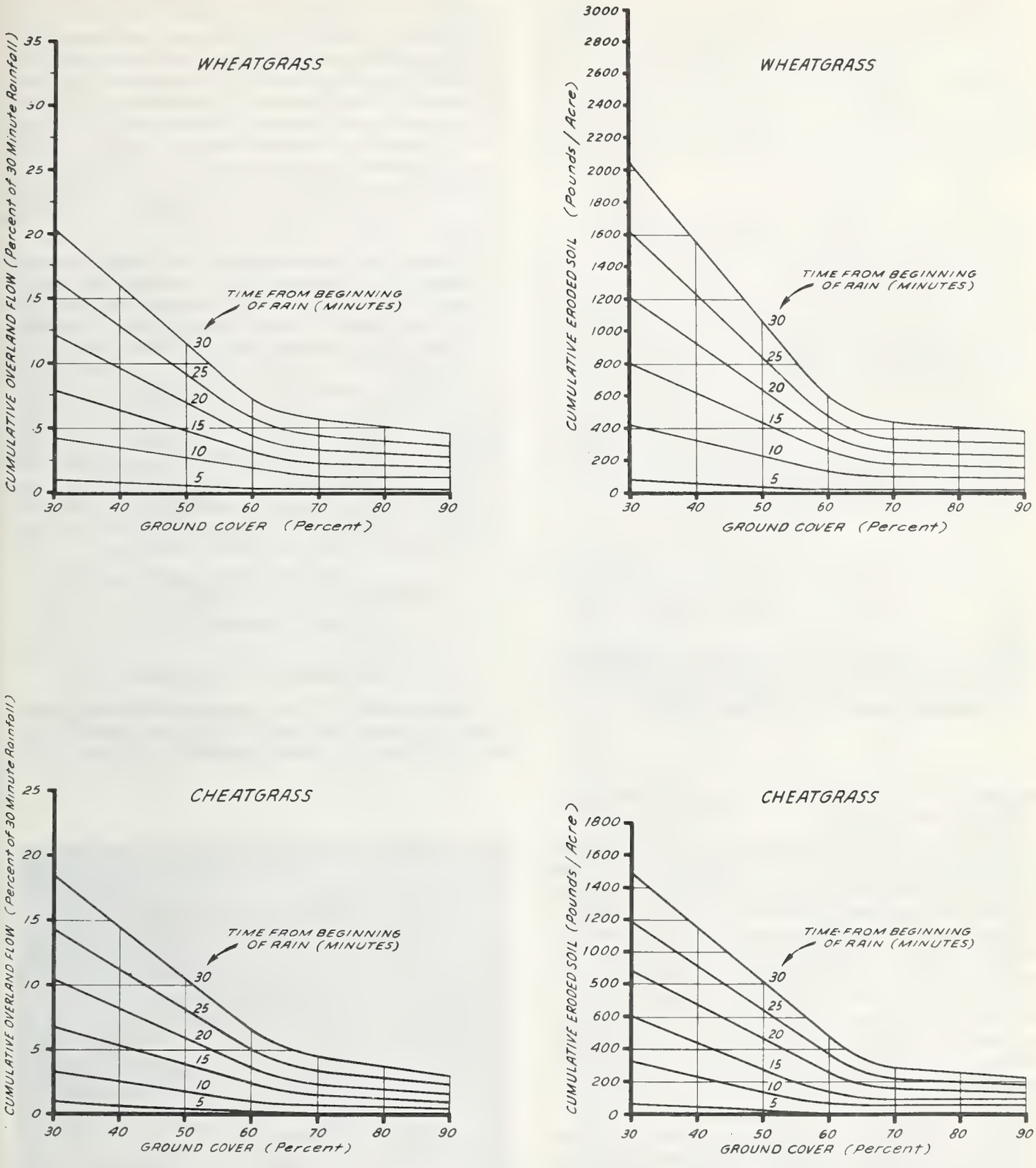

FIGURE 3. — Cumulative overland flow and soil erosion from wheatgrass and cheatgrass sites on soil derived from granite at different ground cover densities by successive 5-minute periods. Based on a simulated rainfall of 1.87 inches for 30 minutes (*12*).

Contour Trenching

The principle of contour trenching as a soil conservation measure is very old, but the use of trenches to secure soil stabilization on high

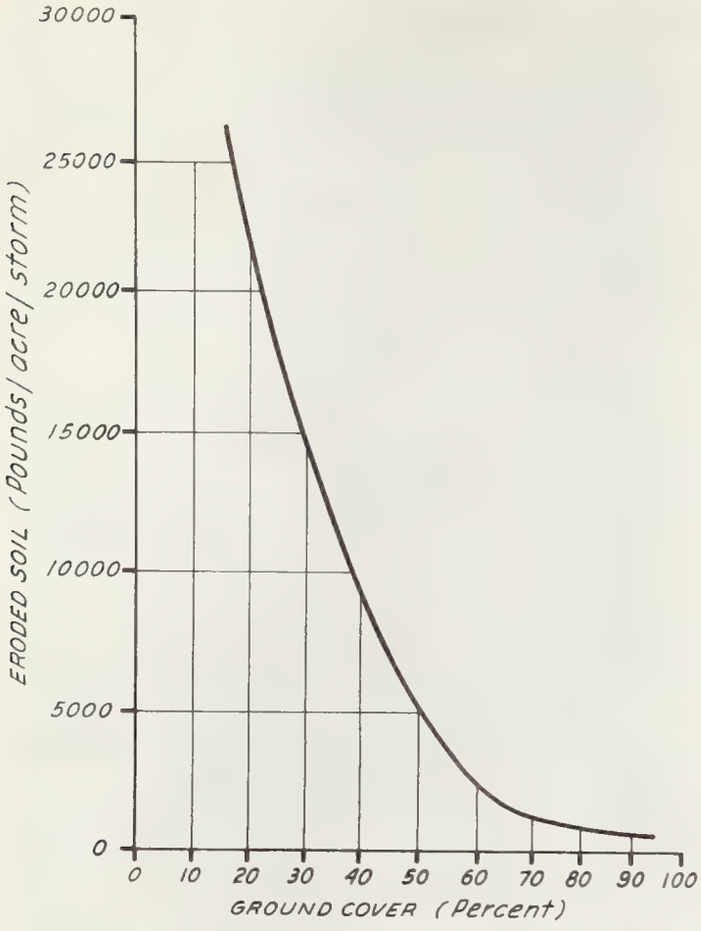

FIGURE 4. — Relation of eroded soil to ground cover density on experimental watershed A, subalpine-herbaceous range, Utah. Based on simulated rainfall of 1.5 inches per hour for 20 minutes (*10*).

mountain watersheds and to prevent floods is relatively new (*11*).

In 1933 the Intermountain Forest and Range Experiment Station, U.S. Forest Service, was given the responsibility of devising the most efficient and effective methods for the control of floods and erosion originating on forest and range lands (*2*). Development of the trenching system to get water into the ground where it falls, thus preventing surface runoff, erosion, and sedimentation, has proved most effective. In addition, the application creates favorable moisture conditions in the soil that hasten the restoration of plant cover needed to stabilize the soil further.

The principle of contour trenching has proved so sound and effective that several thousand acres of badly eroded flood-source areas in the intermountain west have been treated and many more thousands of acres are scheduled for treatment. Since its inception nearly 30 years ago, the criteria guiding application of the contour trenching system and the construction and design techniques have been greatly improved upon. Some of these criteria are discussed here.

Contour trenching is a precision-type job that requires a combination of close analysis of potential surface runoff, recognition of site adaptability, and careful construction. Failure to recognize or give full consideration to any one of these three items can cause failure of the work. Careful consideration of them can provide complete control of surface runoff and erosion from treated areas occasioned by high-intensity summer rainstorms.

The contour trench system consists of a series of zero-grade insloping-type trenches spaced sufficiently close to hold a predetermined amount of surface runoff. Small check dams or baffles are constructed across the trenches at intervals of about 35 feet to segment them. These baffles are slightly lower than the fill-dike to allow water to flow along the trench without overtopping the trench. Runoff calculations are obtained by using runoff curves developed for different vegetation and soil types (fig. 6).

The following formula relates rainfall, trench spacing, and capacity aids in designing contour trench systems for various conditions:

$$\text{Rainfall} \times \frac{\text{Estimated runoff (percent)}}{100} \times \frac{\text{Trench interval (ft.)}}{12} = \text{Trench capacity (cu. ft./lineal ft.)}$$

Considerable care is needed in selecting sites that are adaptable to the contour trench system. Soil depth, mass stability of soils, rock

FIGURE 5. — Contour trench system installed on a deteriorated watershed for flood prevention and watershed protection purposes. Slope gradients vary from 30 to 70 percent. Evaluations made subsequent to construction indicate the trenching system has materially reduced downstream sedimentation.

outcroppings, and slope gradient all influence the decision of whether to trench or not.

Because the trenching system is a device to hold water until it can be disposed of by infiltration, a minimum of soil depth is necessary. Also, soil is necessary to provide a favorable seedbed for the establishment of vegetation on the soils disturbed by trench construction.

Normally, areas that have 24 to 30 inches of soil development above the unaltered parent material can be trenched successfully.

Mass stability of the soil is another factor determining the applicability of the trenching system. Areas with a history of mass soil movement or slumping should be avoided. Likewise, sites having a substratum of impervious clay soils should be avoided even though slumping is not obvious.

The pattern and disposition of rock outcroppings within the headwaters of a drainage also exert an influence on the decision to trench. Since each trench of the system is in itself a reservoir, the trench pattern should be continuous across the slope and extend to the upper limits of the flood-source area. If rock outcroppings at the head of a drainage are contributing to oversurface flows of water and such outcroppings preclude trench construction, there is a possibility that runoff from the upper slope will exceed the capacity of the trench, causing the trench to fail. A similar situation can occur when working across the slope and rock outcroppings prohibit a continuous run of the trench.

Many areas containing rock outcroppings have been successfully trenched. The main point to remember is that each trench must be capable of storing and disposing of the water deposited into it from the land above. Under no circumstance should water from an upper trench be diverted into a lower trench nor should water from a trench be diverted to a natural channel.

Once it has been determined that a site is adaptable for the contour trenching system, construction can begin. Stages of the construction process are shown in figures 7 to 11.

The construction process, although not com-

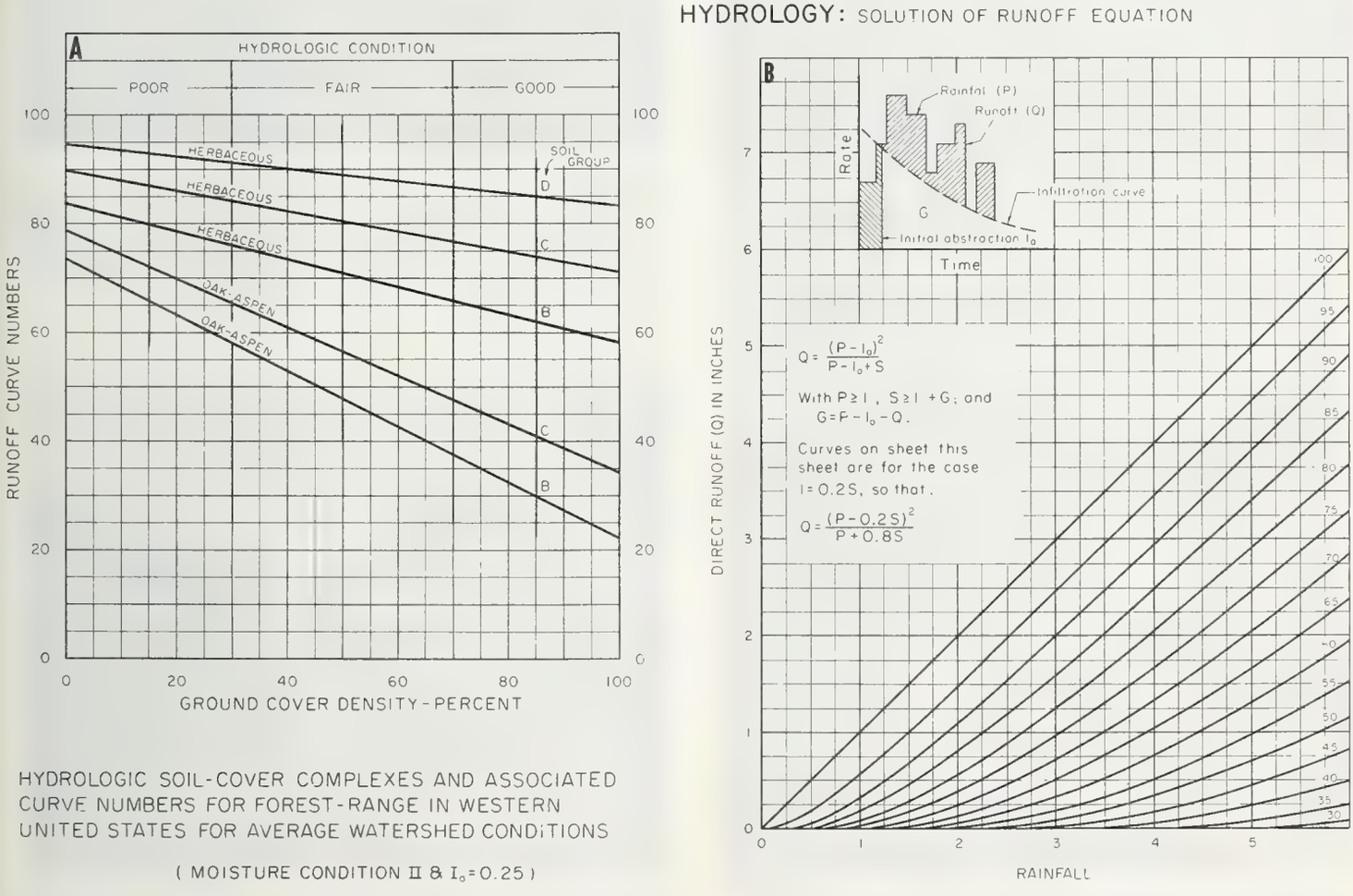

FIGURE 6.—*A*, Based on soil group, vegetal type, and ground cover density, curve number (fig. 6, *B*) is selected for solution of the runoff equation. *B*, With runoff curve number selected from figure 6, *A*, surface runoff can be computed for various rainfall intensities (*13*).

plex, requires an understanding of certain facts involving a knowledge of several skills and disciplines. Failure to apply these skills and disciplines to the specific characteristics of the site to be treated can result in an unsatisfactory job. On the other hand, skillful application can result in a satisfactory job that will provide permanent flood control measures. Some of the more important facets of the construction job are enumerated below:

1. Contour trenching is a precision job, and each trench must be on a zero grade. Accordingly, each trench must be laid out on the contour, with a hand level and common builder's lath for stakes. Stakes must be placed frequently enough that they can be easily followed by the tractor operator and to keep the machine on level grade across undulating terrain. Each high and each low point need be staked to avoid undulations in the trench.

2. The horizontal spacing interval, as determined from the formula, should be fairly

FIGURE 7. — On steep terrain, the first step of contour trench construction is building a level work platform on which the tractor operates. The platform need be no wider than necessary to accommodate the equipment safely.

FIGURE 9. — The trench is constructed along the platform by a cut-and-fill operation. With the use of full angle and tilt of the blade, excavated dirt spills under the outer track of the machine, where it is packed to form the outer dike of the trench.

FIGURE 8. — After the platform is constructed, the upper 2 feet of the steep vertical cutbank are knocked down to aid in backsloping.

FIGURE 10. — Trench on 70-percent slope completed to standard except for cross dams or baffles, which are constructed as the tractor backs out.

FIGURE 11. — Completed trench after construction of cross dams or baffles. Trench has a storage capacity of 7 to 10 cubic feet per lineal foot of trench. Cross dams are lower than trench dike, to allow equalization of stored water in trench and to prevent spill over the trench.

constant. To prevent trenches from converging because of increasing slope gradients, staking should commence on the steepest slope to be treated. If slope gradient lessens and the trench interval increases beyond 30 percent, short trenches will be needed to fill in the gap.

3. Powerful crawler-type tractors with full-angle and tilt dozer blades are essential to successful contour trench construction. If the work is in rocky terrain or in tight soils, a rear-mounted hydraulic ripper is recommended. Because most work is done at high elevations where equipment efficiency is less, tractors with over 140-drawbar horsepower are advisable. Experience gained from high elevation work indicates a lower unit cost per lineal foot of trench when larger size equipment is used.

4. Well-trained and competent equipment operators and job supervisors are essential for a successful operation. Several days of training for new crews by personnel experienced in trenching work are advisable. Inexperience and improper job supervision can result in a situation worse than that originally existing.

5. Live watercourses and seep areas should be avoided. Contour trenches are designed primarily for collecting and disposing of over-the-surface flows of water from rainstorms. They will not function properly if continually saturated from live water sources.

6. All areas of soil disturbance caused by construction activities need to be revegetated with adaptable plant species. The continued effectiveness of the trenching system depends upon stabilizing the cut-and-fill slopes caused by construction as well as the establishment of adequate ground cover between the trenches. In most instances, natural revegetation occurs rapidly between trenches, since soil movement has been arrested and soil moisture conditions improved.

Effectiveness of the Contour Trenching System

A few quantitative and many qualitative evaluations have been made to determine the effectiveness of contour trenching to control flooding (figs. 12 and 13). Perhaps the most exhaus-

FIGURE 12. — Top, severely depleted watershed in Upper Dry Creek, with ground cover density of less than 40 percent in 1957. Bottom, the same area after a high intensity summer rainstorm in 1958. Oversurface flow of water established a gully pattern, concentrated runoff, and contributed to downstream flood and sedimentation.

tive and best instrumented study was conducted in the Davis County Experimental Watershed, Utah, where an intensive system of trenches was constructed in the early 1930's.

The nonflooding behavior of these rehabilitated watersheds has been evaluated (*5*) and is particularly significant when one considers that during two storms greater rainfall rates were attained than had ever been recorded in the State of Utah. During a rainfall of 1.14 inches on July 10, 1936, a rate of 5.04 inches per hour for a 5-minute period was registered. On the evening of August 19, 1945, when 1.09 inches of rain fell, rates at several of the recording gages exceeded 6.00 inches per hour for a 5-minute period, and at one a rate of 6.80 inches per hour was registered (table 1).

The storm on July 10, 1936, produced no floods from the treated watersheds, but the same rainfall caused mud-rock floods in four drainages within the area that had not been treated. Rehabilitated watersheds again on August 19, 1945, when subjected to the unusually high rainfall rate of 6.00 inches per hour, disposed of the precipitation without erosion or runoff of flood proportions.

The findings of the Davis County evaluations have been further corroborated by numerous field investigations of comparable watersheds during and immediately after high intensity rainstorms. In all investigations, treated watersheds were compared with contiguous nontreated watersheds. Duration and intensity of precipitation in each watershed were judged to be comparable. In every case it was found that contour trenches significantly reduced surface

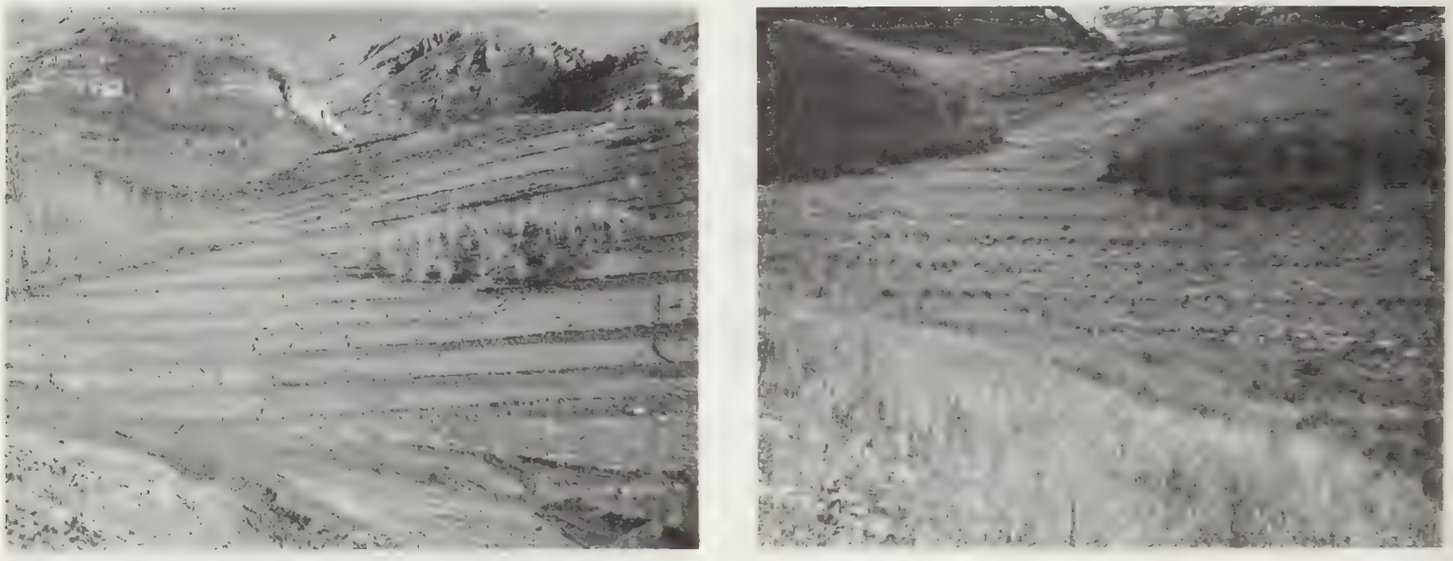

FIGURE 13. — Left, system of contour trenches constructed on Dry Creek flood-source area in fall of 1958. Trenches break up gully pattern. Right, Dry Creek area 3 years after installation of contour trench system (1961). Seeded grasses well established on soils disturbed by construction activities. Improved soil moisture condition between trenches have contributed to reestablishment of native vegetation.

TABLE 1.—*Summer storm rainfall and resultant overland flow and soil losses from Parrish plots, Davis County Experimental Watershed, Utah*

Storm dates	Total rainfall	Nonflood-source area		Flood-source area		Artificially denuded	
		Overland flow	Soil eroded	Overland flow	Soil eroded	Overland flow	Soil eroded
	In.	*Pct.*	*Cu. ft./acre*	*Pct.*	*Cu. ft./acre*	*Pct.*	*Cu. ft./acre*
July 10, 1936	1.14	0.7	0	42.8	181.5	.....	.....
July 16, 1936	.89	.4	0	43.4	153.6	.....	.....
July 28, 1936	1.21	.2	0	33.0	83.2	.....	.....
Aug. 18–20, 1945	3.09	.5	0	24.3	92.8	.....	.....
July 10, 1950	.70	.9	0	12.6	[1]	61.3	215.3
Aug. 19, 1951	1.15	.6	0	8.4	[1]	46.6	186.2
Aug. 4, 1954	1.17	.4	0	3.8	[1]	31.3	91.3
Aug. 19–20, 1959	.98	.6	0	2.3	[1]	43.7	98.4
Sept. 3, 1960	.63	.....	.....	.....	.....	31.0	110.0
Aug. 25, 1961	.64	.....	.....	.....	.....	28.6	89.2
July 13–24, 1962	2.59	.....	.....	.....	.....	39.0	401.3

[1] Trace.

water runoff into stream channels, reduced peak discharge of streams, and effectively reduced soil erosion and resultant downstream sedimentation.

Summary

Several methods of obtaining soil stabilization on high mountain watersheds are available to the land administrator. These are (1) intensive management practices, (2) revegetation coupled with intensive management practice, and (3) contour trenching. Each method recognizes the fundamental relation existing between land cover and hydrologic behavior and reflects the importance of maintaining the productivity of the site for the production of forage, fiber, wildlife, and recreation.

The application of each method requires a careful analysis of the (1) geologic norm, (2) type of flooding, (3) watershed protection requirements, and (4) adaptability of the site for treatment.

Of the methods described, contour trenching has proved most effective in controlling flooding, and sedimentation occurring from badly deteriorated mountain watersheds. The application of this method is not a panacea for all flood-source areas but has proved effective in controlling flooding from badly deteriorated lands occasioned by high intensity summer rainstorms.

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EMERGENCY MEASURES TO CONTROL EROSION AFTER A FIRE ON THE SAN DIMAS EXPERIMENTAL FOREST

[Paper No. 19]

By R. M. Rice, R. P. Crouse, and E. S. Corbett, *research foresters, Pacific Southwest Forest and Range Experiment Station, Forest Service, U.S. Department of Agriculture, Glendora and Berkeley*

Southern California is in urgent need of better solutions to its flood and erosion problems. Its steady population growth has resulted in intensive urban development on the debris cones at the mouths of mountain canyons. The Mediterranean climate, which attracts thousands of newcomers to the State, favors the growth of a highly inflammable vegetative cover (chaparral) on the mountains. It also frequently produces weather conditions that promote widespread wildfires. Such fires damage watersheds on steep mountain slopes that often lie above densely populated cities. This hazard justifies flood and erosion control measures that may not be necessary in most parts of the United States.

The U.S. Forest Service is conducting a broad research program in southern California to test several flood and erosion control measures for

use after fire on mountainside slopes and in small tributary channels. This study is in progress at the San Dimas Experimental Forest, which has been devoted to watershed management research since 1933. Owing to the difficulty of burning watersheds under controlled

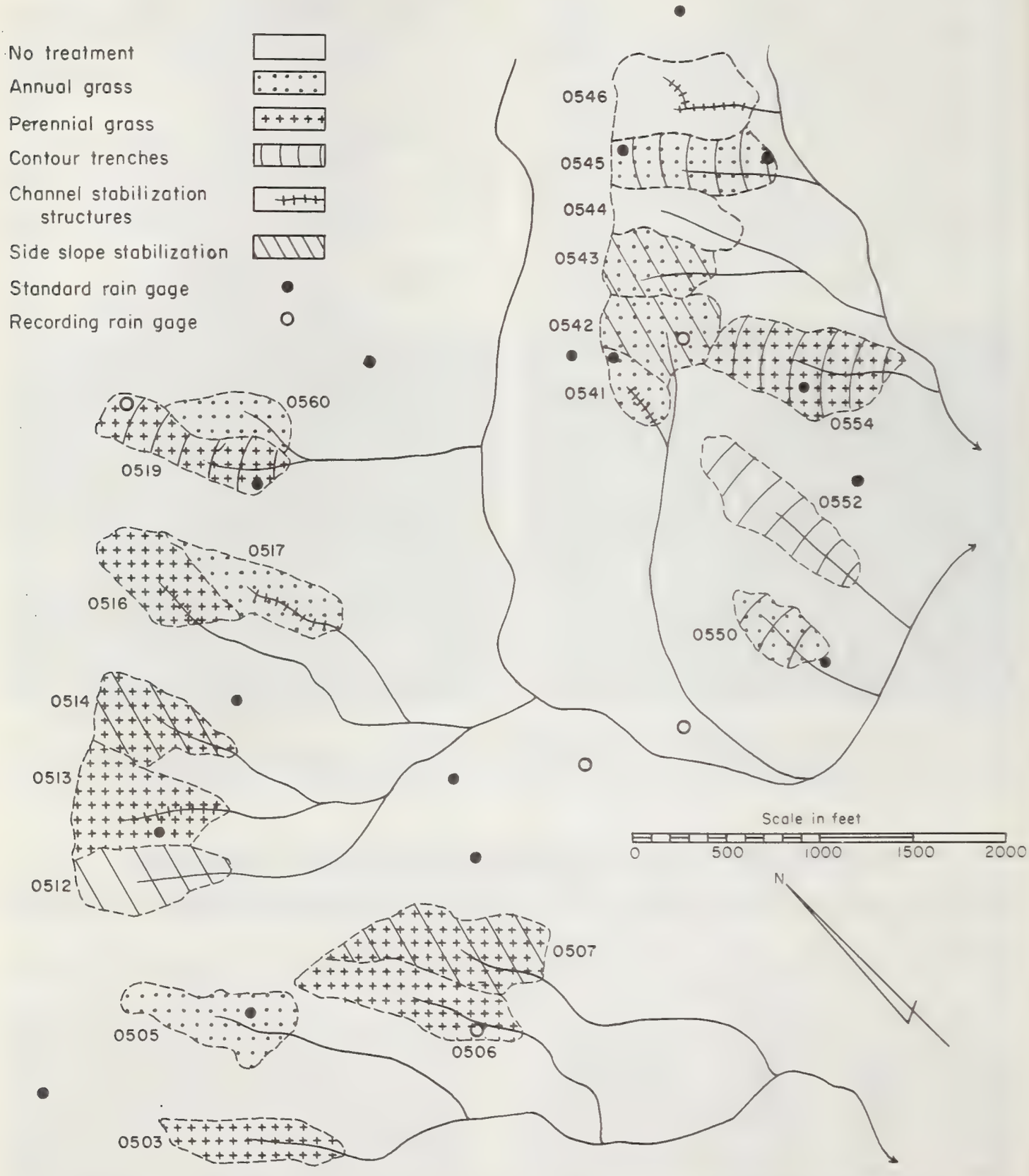

FIGURE 1. — Location of treatments on experimental watersheds testing erosion control measures.

conditions, earlier studies of erosion after fire had been limited to plots or to analyses of the effect of wildfires in the area.

In 1960 a wildfire swept most of the 17,000-acre Experimental Forest. It destroyed much valuable research underway, but it also presented an opportunity to study flood flows and erosion rates from completely burned watersheds. To exploit this opportunity, Federal, State, and county agencies undertook a cooperative emergency research program.

This paper reports on a continuing study begun during the dry winter of 1960-61, and covers data collected during the four major storms of the 1961-62 season.

Methods

We are seeking to obtain a quantitative evaluation of several mechanical and vegetative land treatments as "first aid" for burned chaparral watersheds.

Selection of Watersheds

The 20 watersheds used in this study were chosen to be as similar as possible in size (about 5 acres), shape, aspect, and erodibility (fig. 1). Even so, considerable variation existed within the group. To reduce the effect of this variability upon our study, we grouped watersheds into five erodibility classes based on three independent appraisals of their relative erodibilities.

The appraisers used slope, channel gradient, rockiness, and amount of colluvial soils as indicators of the relative erodibility of the watersheds. Treatments were then assigned to the watersheds so that apparent erodibility was balanced among treatments; that is, no treatment could be applied to all the high erodibility groups.

Instrumentation

The streamflow from each experimental watershed is measured in a 30-foot trapezoidal flume. Debris is trapped and measured behind earth-fill dams that can hold about 60 cubic yards per acre. Precipitation is measured in 5 intensity rain gages and 16 nonrecording gages distributed throughout the study area.

Selection of Treatments

In southern California broadcast sowing of annual grasses is the most widely used method of rehabilitating watersheds after a fire. Eight of the watersheds were sown to a mixture of annual grasses (Wimmera 62 ryegrass, *Lolium rigidum:* and blando brome, *Bromus mollis*) — four at the rate of 2.5 pounds per acre and four at 20 pounds per acre.

Perennial grasses — often suggested for use — were also tried. Eight watersheds were seeded to a mixture of perennial grasses (intermediate wheatgrass, *Agropyron intermedium;* pubescent wheatgrass, *A. trichophorum;* tall wheatgrass, *A. elongatum;* hardingrass, *Phalaris tuberosa stenoptera;* big bluegrass, *Poa ampla;* smilograss, *Oryzopsis miliacea*) and small amounts of the annuals, blando brome and Wimmera ryegrass — four at the rate of 4.5 pounds per acre and four at 20 pounds per acre. Four watersheds were left unseeded.

The areas sown to perennials were sprayed with 2,4-D and 2,4,5-T during the springs of 1961 and 1962 to help establish perennial grass by reducing competition from brush species.

Precipitation after sowing was light (total of 6.29 inches, 16 storms) during the 1960-61 season. Consequently, we had scant cover from seeded species. To correct this shortage, broadcast sowings were repeated in the fall of 1961.

In addition to the broadcast sowings, three mechanical erosion control measures were being tested. Chosen from measures currently in use in Western United States, each treatment combats the movement of water and soil at a different place along the route from raindrop impact to the debris basin. These mechanical treatments were distributed among the watersheds orthogonal to the broadcast sowings (see tables 2 and 3).

Side slope stabilization. — This treatment consisted of planting barley and fertilizer in hand-hoed rows at 2-foot intervals on the contour (150 pounds of barley and 140 pounds of diammonium phosphate per acre). Its objective was to create closely spaced barriers to the overland flow of water and debris. This treatment was compared with other mechanical measures because we were merely using plants as a means of obtaining the desired pattern of obstructions. The barley plants undoubtedly also promoted infiltration and reduced rainfall impact.

Contour trenching. — This method is being used in Idaho and Utah under somewhat different conditions of climate and soil. It has the effect of breaking up surface flow, increasing depression storage, encouraging the infiltration of storm runoff, and trapping sediment and debris. In this study the trenches were put in as close together as the terrain would permit (40 feet on the gentler slopes to 90 feet on the steeper slopes). The trenches provided storage for about 3 inches of rainfall. Storms of large size or of very high intensity may overtop the trenches. In order to provide for this situation, each trench was drained either into the stream

channel or into another trench below. This drainage system consisted of about six 12-inch half-round downspouts for each treated watershed.

Channel stabilization. — Channel stabilization was attempted by building small gravity channel check dams from soil cement. In the watersheds so treated, a system embodying both natural and artificial controls was designed to stabilize the channel in those portions with less than 30 percent normal gradient. This treatment attempts to lessen channel downcutting and, thereby, helps stabilize the toe of colluvial soils resting on side slopes.

The Analysis

A multiple linear regression model was used in analyzing the data. Nine separate analyses were made — each based on the flood peaks or debris production of an individual storm, except one analysis that used total annual debris production as a dependent variable. Several continuous variables were included in analyses in addition to the class variables used to express treatment effects. The vegetation variable expressed differences in natural vegetative recovery of the watersheds. The other continuous variables gave a further description of the inherent differences of the watersheds. The model tested was:

$$Y=a+\sum_{i=1}^{4} b_iM_i+\sum_{i=4}^{9} b_iV_i+b_{10}N+b_{11}\mathrm{R}+b_{12}S+b_{13}C+b_{14}A$$

in which Y is the dependent variable expressed as cubic feet per second per acre for flood peaks or cubic yards per acre for debris production.

a is the mean response of the experimental watersheds.

M_i are the four mechanical treatment variables taking a value of 0 or 1, depending on the presence or absence of the individual treatments.

V_i are the five vegetative treatment variables treated in the same manner as the M_i.

N is the vegetative cover of the watersheds due to the residual native vegetation and the recovery of native plants (includes burned brush stems and litter). The cover was obtained by using visual estimates of cover on clusters of four 1-square-foot quadrats distributed in a stratified (on aspect) random fashion in the watersheds. About 140 square feet of each watershed was sampled. Expressed as a percent, the native plant cover at the time of the first two storms ranged from 3.7 to 16.5 percent, with a mean value of 7.1 percent. Estimated native cover for the third and fourth storms ranged from 2.3 to 29.9 percent, with a mean of 13.2 percent. This variable was included to allow for differences in natural vegetative recovery of the watersheds.

R is the percent of the soil surface covered by rocks greater than one-half inch in diameter. This variable ranged in value from 0.4 to 15.3. The mean was 7.4 percent. This variable was intended to index the effect that armoring the surface of the watershed with rocks might have on erodibility and storm runoff.

S is the mean slope of the watershed as determined by averaging the slopes at the vegetative sampling plots. This variable ranged from 37 to 69 percent with an average of 54 percent.

C is the mean channel gradient of the watersheds measured from 1:4000 scale aerial photographs. Gradients vary from 17 to 44 percent with an average of 27 percent.

A is the area of the watershed above the flume for flood peak analyses and the area above the debris dam for debris production analyses. The average area above the flumes is 4.67 acres (range: 1.38 to 7.31 acres). The average area above the debris dam is 5.68 acres (range: 2.26 to 9.57). As Anderson reiterated in a recent paper "most of the variables which we neglect to put in our analyses and many of our mistakes in choice of functions hide in the area variable."[1] We included area in our analyses as index-of-ignorance variable. In each of the regression analyses four models were tested for each set of dependent variables: The first analysis included all independent variables; the second omitted the continuous variable; the third omitted the vegetative treatment variable; and the fourth omitted the mechanical treatment variables.

[1] ANDERSON, H. W. A MODEL FOR EVALUATING WILDLAND MANAGEMENT FOR FLOOD PREVENTION. Pacific Southwest Forest and Range Expt. Sta. Tech. Paper 69, 12 pp. 1962.

Results [2]

Storms

Four of the 22 storms during the 1961-62 season were long and intense enough to produce responses in the experimental watersheds that could be analyzed (table 1). Analysis of the

TABLE 1.—*Rainfall intensities and amounts for major storms of hydrologic year 1962* [1]

Storm date	Maximum rainfall intensity for duration of—						Total storm precipitation
	5 min.	10 min.	15 min.	20 min.	30 min.	60 min.	
	In./hr.	*In./hr.*	*In./hr.*	*In./hr.*	*In./hr.*	*In./hr.*	*Inches*
Nov. 20, 1961..	2.16	1.70	1.38	1.21	1.16	.99	2.47
Nov. 30 to Dec. 3, 1961...	1.80	1.32	1.09	.96	.91	.73	4.58
Jan. 20 to Jan. 23, 1962..	1.38	1.10	.92	.82	.79	.70	4.75
Feb. 7 to Feb. 12, 1962..	1.59	1.24	.96	.81	.63	.39	9.27

[1]Average of gages in vicinity of study watersheds.

catch of rain gages indicated there were no appreciable differences in the amounts or intensities of rainfall among the watersheds. Consequently, rainfall variables were not included in the regression analyses. The average concentration time of the watersheds is about 8 minutes. The 10-minute intensities shown in table 1 have recurrence intervals of 2.1 years, 1.1 years, 0.5 year, and 0.8 year. The recurrence intervals for the maximum 24-hour precipitation are 0.5 year, 0.6 year, and 1.0 year.

Storm of November 20, 1961

The first storm of the season had rainfall of high intensities. This downpour resulted in high flood peaks and heavy debris from the watersheds with no mechanical treatment and in moderately high peaks and moderate amounts of debris from watersheds modified by channel, check dams, or treated side slopes. The responses were small from the contour-trenched watersheds, because the storm failed to exceed the storage capacity of the trenches. During this storm, the channel-stabilizing dams continued to be filled with debris. Thus, additional channel storage was available that, consequently, reduced the flood crests of these watersheds.

Storm of November 30 to December 3, 1961

During the second storm of the season, most of the channel structures filled with debris to the level of the spillway. This storm was the first since the study was begun to exceed the storage capacity of contour trenches. Several contour trenches failed, causing an increase in flood peaks and debris production from these watersheds, although rainfall was less intense during this storm (table 1).

Storm of January 20 to 23, 1962

The third storm was the gentlest and failed to reveal any dramatic effects of treatment differences. The general relations between mechanical treatments appear to be continuing.

Storm of February 7 to 12, 1962

The last storm of the season lasted 5 days and provided about two to four times as much precipitation as any other storm (table 1). During most of the storm, the rain fell at relatively low intensities (0.25 in./hr.). The peak flows were in response to short bursts of high-intensity rainfall on the fourth day of the storm.

Additional contour trenches failed during this storm, because storage capacity was greatly exceeded. With rare exceptions check dams that stabilized channels were filled with debris. In many cases, debris cones extend upstream to the toe of the next higher dam, indicating that the design profile had been reached.

The channel-stabilized watersheds had the highest flood peaks, continuing what appears to be a trend toward higher relative crests in comparison with the other treatments. The watersheds with stabilized side slopes, on the other hand, seem to be maintaining lower relative peaks. In this storm their average was about two-thirds the average of all the other watersheds.[3]

Seasonal Results

Two measures of practical importance to the land manager are the total annual production of debris (the debris to be disposed of, table 2) and the highest flood peak for the year (the peak to be guarded against, table 3).

As the row means in tables 2 and 3 show, watersheds seeded to low density perennial grasses had higher peaks and heavier debris production. Debris production appears to have been reduced by the high density sowings. But whether this is a true effect is questionable, because the vegetative cover from seeded grasses in all watersheds was very low. The best cover was obtained in the watersheds having high density ryegrass where the seeded annual grass cover amounted to 1.8 percent in the spring of

[2] The authors wish to acknowledge the assistance of Donald W. Seegrist, statistician, Pacific Southwest Forest and Range Experiment Station, Berkeley, Calif., for conducting the regression analyses and statistical tests.

[3] Detailed data on flood peaks and debris production for each storm are available from the Pacific Southwest Forest and Range Experiment Station, Forest Service, U.S. Department of Agriculture, 110 North Wabash Avenue, Glendora, Calif.

TABLE 2.—*Total debris production by treatments during hydrologic year 1962*

Broadcast sowing treatments	Mechanical treatments				
	No mechanical treatment	Contour trenches	Channel stabilization	Side slope stabilization	Mean [1] response
	Cu.yd./acre	*Cu.yd./acre*	*Cu.yd./acre*	*Cu.yd./acre*	*Cu.yd./acre*
No broadcast seeding	29.0	16.7	13.8	9.9	17.4
Low density annual grass	39.7	1.8	25.4	5.8	18.2
High density annual grass	31.4	6.7	8.4	9.1	13.9
Low density perennial grass	35.6	26.6	26.4	8.6	24.3
High density perennial grass	26.0	6.9	23.3	8.7	16.2
Mean response [2]	32.3	11.7	19.5	8.4	18.0

[1] Row effects significant at the 0.23 level.
[2] Column effects significant at the 0.03 level.

TABLE 3.—*Highest flood peaks by treatments observed during hydrologic year 1962*

Broadcast sowing treatments	Mechanical treatments				
	No mechanical treatment	Contour trenches	Channel stabilization	Side slope stabilization	Mean [1] response
	Cu.ft./sec./acre	*Cu.ft./sec./acre*	*Cu.ft./sec./acre*	*Cu.ft./sec./acre*	*Cu.ft./sec./acre*
No broadcast seeding	2.1	1.7	1.2	3.8	2.2
Low density annual grass	6.0	1.6	2.8	1.1	2.9
High density annual grass	2.5	1.4	3.3	1.8	2.2
Low density perennial grass	3.6	7.4	7.7	2.5	5.3
High density perennial grass	3.9	0.7	3.0	1.8	2.4
Mean response [2]	3.6	2.6	3.6	2.2	3.0

[1] Row effects significant at the 0.23 level.
[2] Column effects significant at the 0.64 level.

1961 and 9.9 percent in the spring of 1962. Average total vegetative cover, exclusive of barley, on all watersheds amounted to 7.7 percent and 17.3 percent for the same periods, the majority of it being native species. In watersheds with the side-slope treatment, barley accounted for another 7.0 percent of cover in the spring of 1961 and 13.9 percent in the spring of 1962.

The mechanical treatments produced more striking contrasts, particularly in debris production. Debris yield from the watersheds with no mechanical treatment averaged about 30 cubic yards per acre when allowance is made for the effect of the continuous variables (table 4). The side-slope-stabilized watersheds yielded about 35 percent of this amount, the contour-trenched watersheds 40 percent, and the channel-stabilized watersheds 65 percent. The highest peaks (table 3) form two groupings according to mechanical treatment: (1) those from watersheds with side-slope control (contour trenches and furrow planting of barley), and (2) those from watersheds without control (channel stabilization and no mechanical treatment). This grouping held during each of the four storms studied.

The regression equations calculated from each of these two analyses are shown in table 4.

The debris regression accounted for 90 percent of the variability in the data (i.e., $R^2=0.90$) and the other regression accounted for 74 percent of the variability in peak flow. Figures 2 and 3 illustrate the precision of our prediction equations.

TABLE 4.—*Regression equations for predicting total annual debris production and highest annual flood peak*

Total debris regression coefficients	Peak flow regression coefficients	Independent variables
Cu. yd./acre	*C.f.s./acre*	
$Y=+7.48$	$Y=-2.31$	Constant.
$+11.99$	$+0.57$	If no physical treatment.
-5.96	-0.45	If contour trenched.
$+1.40$	$+0.78$	If channel stabilized.
-7.42	-0.90	If side slope stabilized.
-1.75	-0.82	If no broadcast sowing.
$+1.51$	$+0.10$	If sown to low density annual grasses.
-4.27	-0.60	If sown to high density annual grasses.
$+7.03$	$+2.16$	If sown to low density perennial grasses.
-2.53	-0.83	If sown to high density perennial grasses.
$+0.04x$	$-0.08x$	Native vegetative cover.
$+0.78x$	$+0.10x$	Exposed rock.
$+0.35x$	$+0.07x$	Slope.
$-0.37x$	$+0.09x$	Channel gradient.
$-0.73x$	$-0.18x$	Area.

Discussion and Conclusions

The perennial grass seeding appears to be unsuited to our soils and climate. The poor establishment of perennial grass (never more than 1.7 percent cover in this experiment) could not offset the reduction in native cover, owing to herbicidal chemicals needed to assist that establishment. Watersheds seeded by low

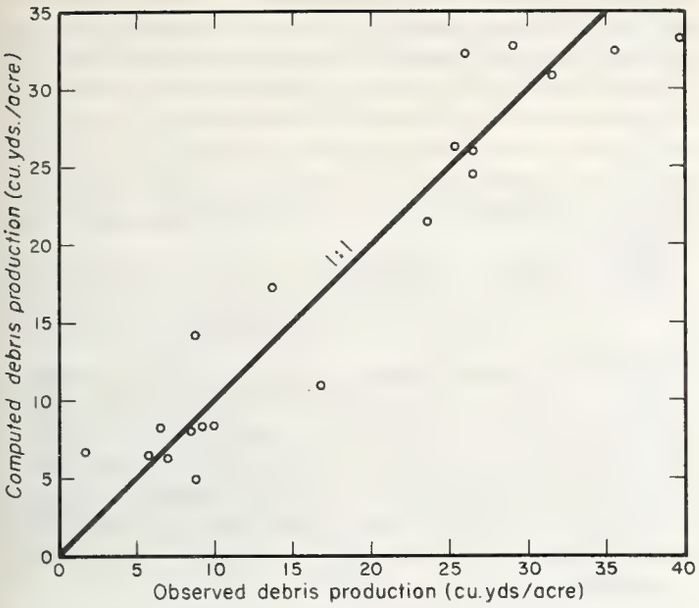

FIGURE 2. — Actual vs. predicted annual debris production (1962).

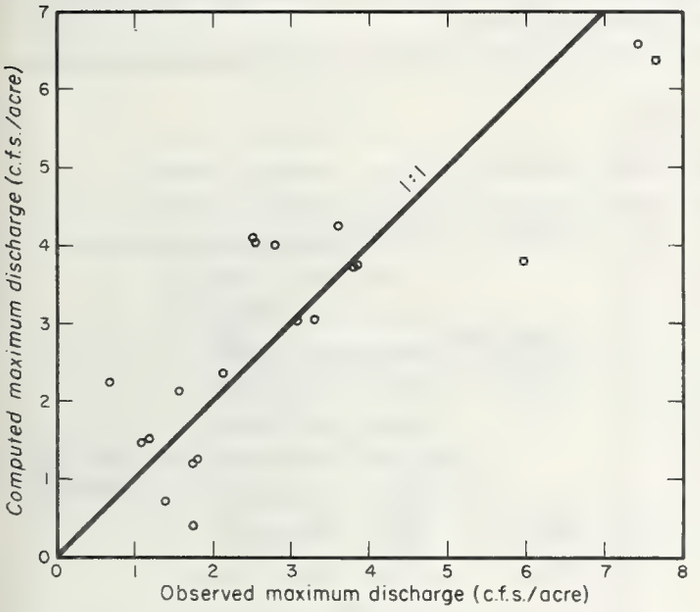

FIGURE 3. — Actual vs. predicted highest annual flood peak.

density perennials tended to have higher flood peaks and greater debris yields than the other watersheds.

Annual grass seeding, on the other hand, may be justified as an emergency erosion control measure. It had no apparent effect on flood peaks, but may have helped reduce debris. Due to the low cost of this treatment (about 1/100 the cost of the mechanical treatments) and the ease and speed with which it can be applied, land managers appear to be justified in gambling on the success of the grass crop.

But there are difficulties in relying solely on broadcast seeding for erosion control in arid regions. To grow a grass crop, an area must have enough rainfall, properly distributed. Storms must be of several days' duration for initial establishment of grass and spaced every few weeks for continued growth. Dry autumn winds, common to this area, can disperse the seed, causing a patchy catch. Also, the grass rarely provides enough cover to do much good during the first year.

We found that the first year's seeding was almost a complete failure. We reseeded the watersheds in the fall of 1961 and had a respectable response by the third and fourth storms. By July 1962, the watersheds of high density annual grass had an average cover of 10.3 percent. The seed produced by this grass promises greater effectiveness from the treatment during the coming season than past performance.

Contour trenches do not appear suited to the terrain and climate of the test watersheds, although the rather limited data do not clearly reflect this fact. By the end of the 1962 season all watersheds had several broken trenches, with little likelihood that the eroding breaks could heal before they produce large amounts of debris. The trenches were necessarily underdesigned because of the steepness of the watersheds. That is, they were not spaced closely enough to provide sufficient storage for the runoff from larger storms. The test results should not be interpreted to mean that contour trenches would be ineffective if they had adequate storage. Our experience during the November 20 storm indicates that closely spaced trenches can be eminently successful in controlling erosion and in reducing peak discharge.

While trenching may not be a dependable control method under our conditions, an increase in depression storage on the side slopes would be highly effective if a method were used that was not so subject to failure. For this reason the trenches in three watersheds were repaired and strengthened so that the effects of increasing side-slope storage could be studied further in future seasons.

The reduction in debris obtained in the channel-stabilized watersheds is greater than the amount currently used to compute cost/benefit ratios for projects with this erosion control measure. However, the associated higher flood peaks argue for a closer look at this erosion control measure.

In our small watersheds, the increase in peaks may come from reduced channel roughness and shorter channel lengths caused by debris filling the rough crooked channels between the check dams. This effect would probably not be so pronounced in larger watersheds. In our case, how-

ever, the storage of debris behind the stabilizing dams during this season was probably a much larger fraction of the total debris produced than in large watersheds. These ambiguities point to a need for further study of this treatment, particularly in larger watersheds.

Side-slope stabilizing by contour furrow planting appear to be the most effective erosion control measure. But the expected effective life of this treatment is only about 4 years. Also, side-slope stabilization is difficult to establish on rapidly eroding areas, such as dry erosion chutes or steep faces undergoing rapid weathering. However, in the majority of cases these limitations may not be serious. Usually most of the area in any watershed can be treated. Using the estimates of Rowe, Countryman, and Storey [4] of debris after fire, with a recurrence of fire in 30 years, we find that 67 percent of a watershed's erosion takes place during the 4 years that the treatment for side-slope stabilization usually persists under conditions in southern California.

The relative superiority of the side-slope-stabilized and contour-trenched watersheds over the channel-stabilized watersheds supports the thesis that, immediately after a fire at least, erosion control measures that prevent the concentration of water or debris are most effective. Barley planted to help stabilize side slopes also provides considerable protection against raindrop impact. From these tests we conclude that preventing the initiation of erosion is the key to postfire erosion control.

[4] ROWE, P. B., COUNTRYMAN, C. M., and STOREY, H. C. HYDROLOGIC ANALYSIS USED TO DETERMINE EFFECTS OF FIRE ON PEAK DISCHARGE AND EROSION RATES IN SOUTHERN CALIFORNIA WATERSHEDS. U.S. Forest Service Pacific Southwest Forest and Range Expt. Sta., 49 pp., illus. 1954.

VEGETATIVE CONTROL OF STREAMBANK EROSION

[Paper No. 20]

By DONALD A. PARSONS, *hydraulic engineer, USDA Sedimentation Laboratory, Soil and Water Conservation Research Division, Agricultural Research Service* [1]

Vegetation may protect a streambank in at least three ways. Perhaps the most important of these is the reduction of water speeds and tractive forces at the soil surface to a value below that required to cause erosion. Second in importance is, perhaps, the protection given to the bank material as a buffer against ice, logs, and other transported materials. The stalwart barrier of trees standing along the edge of a stream prevents the impact of the transported materials with the soft material of the bank. Or, in another way, the tough but pliant shrub-type materials, bending with the forces involved, act as skid surfaces for the transported materials as they are deflected by the banks of streams of all sizes.

Third, close-growing vegetation will contribute to bank stability, within a narrow range of conditions, by inducing deposition. Subsequent to a rare flood that has caused damage but not complete destruction to the vegetative cover, the deposition that occurs in minor floods helps to maintain the bank.

Since the ability of the flood flows to erode the boundary is related to the water speeds near the boundary, it is instructive to measure velocity variations in a vegetated waterway. W. O. Ree (*6*) did this for 8-inch long, dormant bermudagrass. His diagram is reproduced in figure 1, along with the velocity distribution in a comparable, uniform bare channel. The vegetation markedly reduces the water speeds near the soil surface. The rate of change of velocity with distance from the boundary also is less near the boundary with vegetation than without, indicating that the fluid shear stress at the boundary is therefore lower.

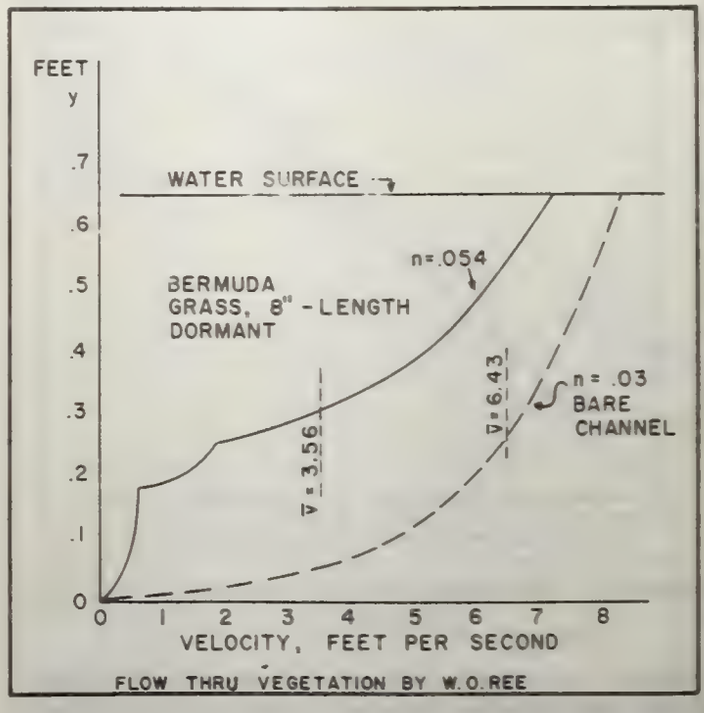

FIGURE 1.—Flow through vegetation (after W. O. Ree).

These low rates of change of velocity near the ground surface and the higher rates of change

[1] Research in cooperation with the University of Mississippi and Mississippi State University.

above the bent, waving grass suggest that the drag or tractive force is divided into two parts. The smaller part in this instance is at the ground surface; the larger part is on the grass tops, transmitted through the stems to the soil mass in contact with the roots.

These results and concepts are important to the understanding of the true functions of vegetative covers. Such an understanding permits a determination of the qualities that suitable vegetative materials should have.

Lessons From the Spartanburg, S.C., Data

As a further aid to judgment of the pertinent qualities of vegetation, an analysis is here made of data from bermudagrass channel flows at the Outdoor Hydraulic Laboratory operated by the Soil Conservation Service near Spartanburg, S.C., for the period 1936-41.

The conditions of the vegetation, the channel geometry, and the hydraulic features of the tests have been published by Ree and Palmer (*7*), but the author has taken the liberty of making the following additional interpretations of these data.

Figures 2 and 3 demonstrate that an empirical relationship of variables is found that well represents most of the flows for widely differing channel and vegetative conditions. The formula is for Chezy's C.

$$C=23 \log Re+A \log \frac{hc}{6500}-98 \qquad (1)$$

$V=C(RS)^{1/2}$=mean velocity; R=hydraulic radius; S=energy gradient; $Re=VR/\nu$=Reynolds number; h=average length of the vegetation top growth, inches; c=the number of stems per square foot; A is a parameter dependent upon the stage of growth and other conditions of the vegetation. Inclusion of the Reynolds number implies that viscosity was a factor affecting the depth and velocity. This is not known, and the data are hardly suitable for determining this.

Figure 4 shows the influence of the total length of bermudagrass per square foot, hc, on Chezy's C, and the manner of derivation of A. Figure 5 shows the variation in A with time of year and state of the vegetation. The retarding effect of the vegetation and therefore its ability, in part, to protect the soil surface, is shown by these two figures to be dependent upon the height and count of the grass and upon whether

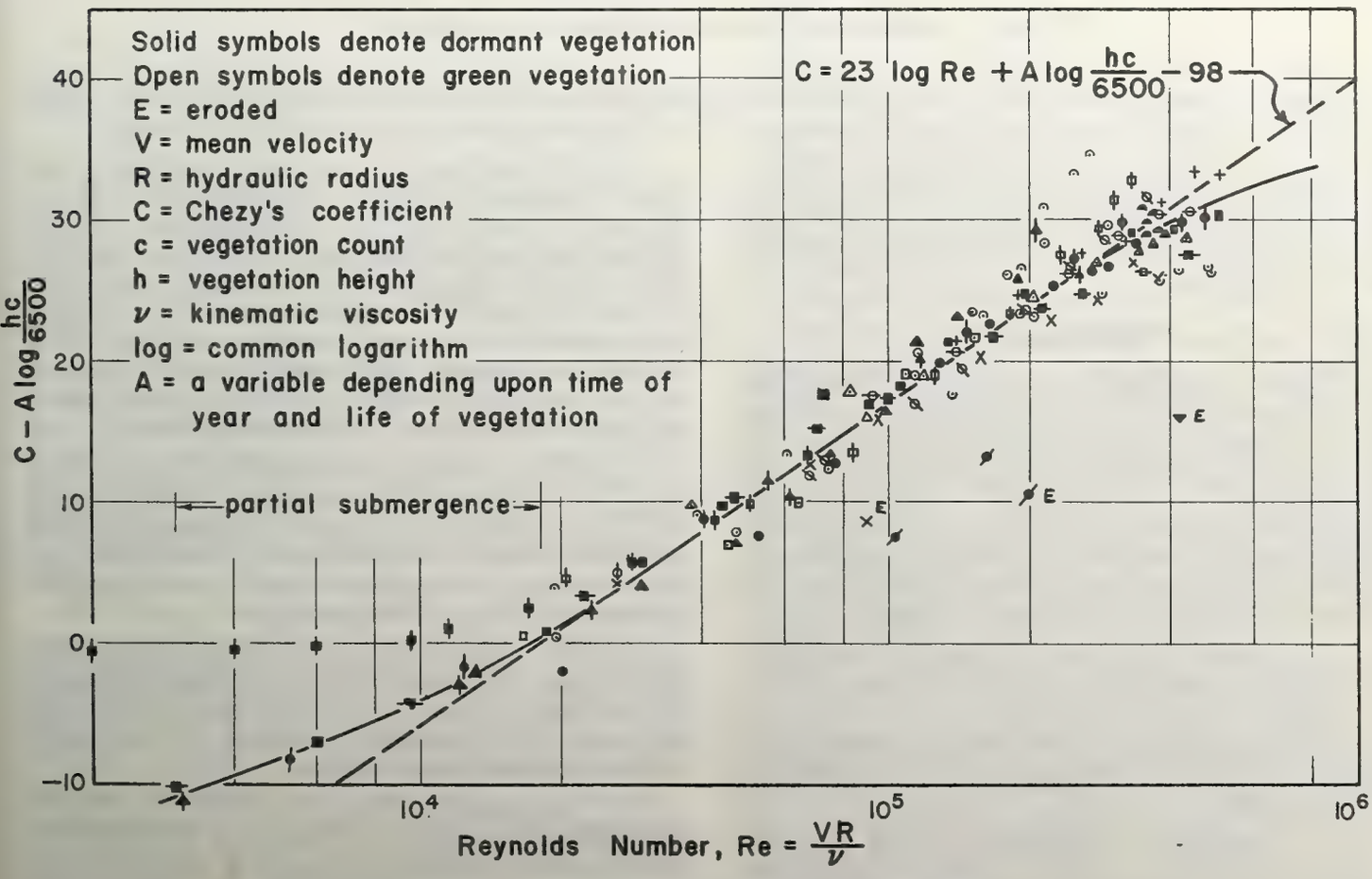

FIGURE 2. — Bermudagrass channels, Spartanburg, S. C.

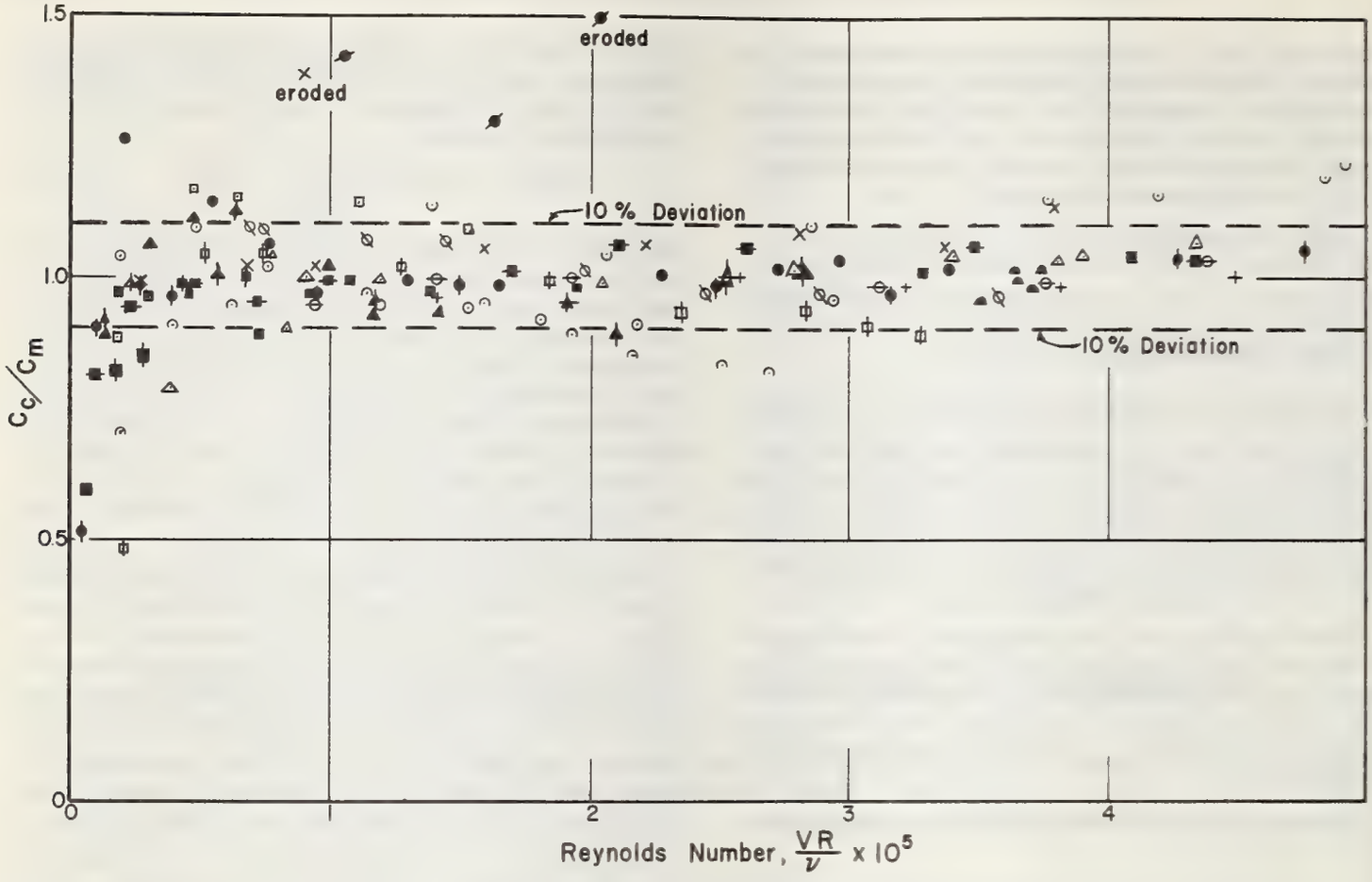

FIGURE 3. — Ratio of calculated to measured values of Chezy's *C*.

the grass is green or dormant. Figure 5 also shows that, other things being equal, the vegetation is most effective when it is young, sturdy, and resilient. After it is killed in the fall, a progressive decrease in effectiveness occurs with time. Its effectiveness is least in late winter and early spring.

Observations

Although there may not yet be good quantitative measurements of the effectiveness of vegetative linings, some things can be stated that are helpful to a better understanding of the use of vegetation in streambank protection work. These, for the most part, are elementary facts and deductions derived from extensive observations over several years. Some come from the findings of hydraulic laboratory tests.

(1) Streambanks that are eroding and need protective measures have little, if any, vegetation on the face of the bank.

(2) If vegetation is to be used to protect such a bank, it must be developed on the bank face between floods.

(3) To establish vegetation quickly, the conditions for growth must be good. Fertilization, mulching, and watering are often needed. Sterile, compact subsoil materials should be scarified and mixed with productive materials.

(4) The portion of an actively eroding bank that is under prolonged submergence is not a favorable place for the development of any known vegetative material. It follows that this part of the bank should be protected by other means. A possible exception would be for shallow depths in very small channels of low velocity where exposed roots and overhanging tops may be capable of reducing the boundary velocity below that needed to scour bare soil.

(5) Since time is required for the development of resistance by vegetation, there is risk of failure because of floods, unless temporary auxiliary protective measures are employed, as, for example, the anchored brush mats along the Winooski River in Vermont (*2*).

(6) The risk of failure during the development stage decreases with increasing elevation on the bank, growth conditions being the same, because of the probable increase in the period before submergence occurs.

(7) The length of time required for effective establishment and development of resistance varies greatly among the kinds and varieties of vegetation. Woody materials are slower than

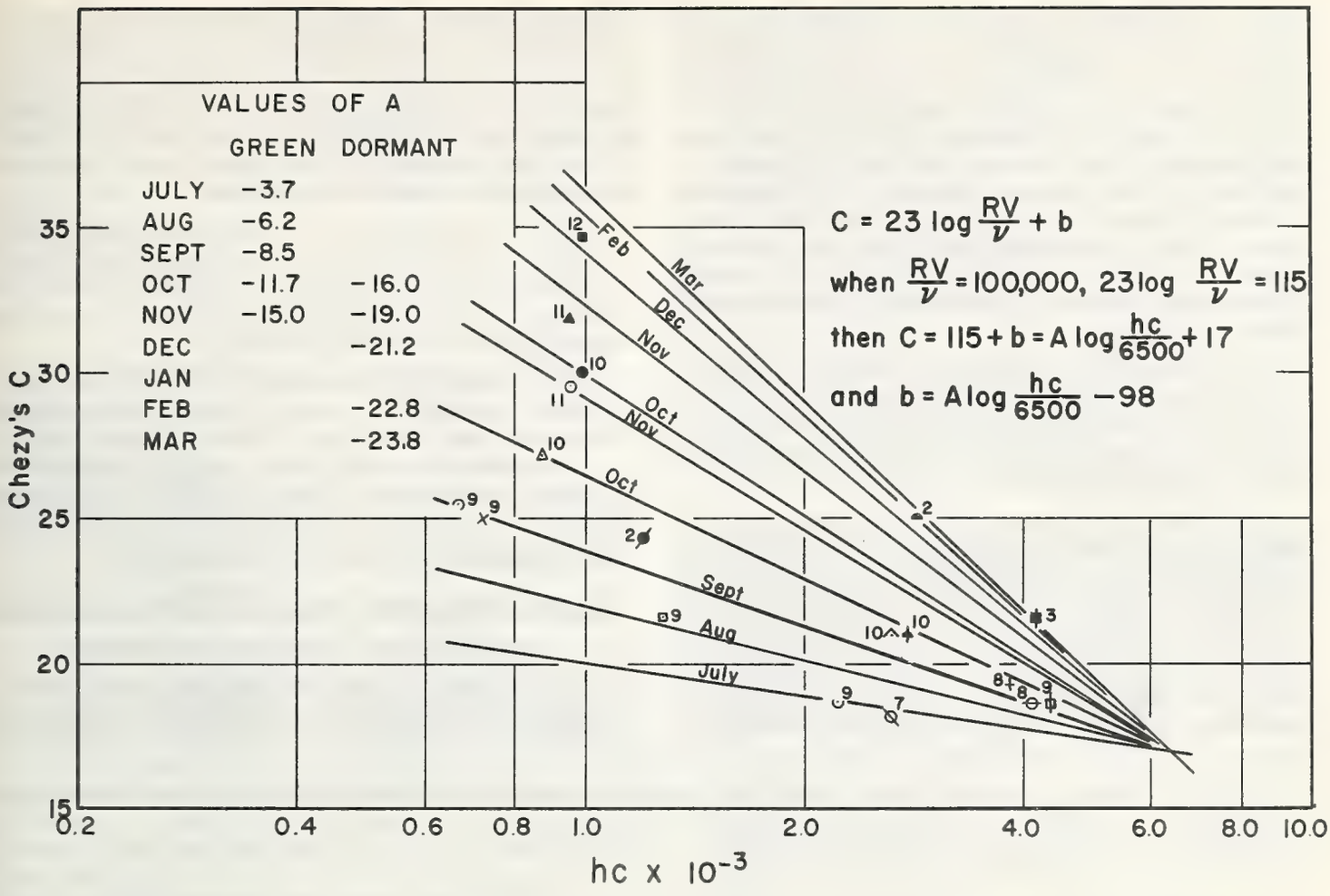

FIGURE 4. — Variation of Chezy's C with vegetation count and height at $Re = VR/\nu = 100{,}000$ (Spartanburg bermudagrass channels).

the grasses. Grass mixtures should include a variety that is quick to become effective, even though at maturity it may not be the most desirable. In the Northeast, an addition of ryegrass to the seed mixture has been good for this purpose.

(8) Since the dependence upon a vegetative lining for protection implies having it on the bank at the time of the flood, the time of its initiation (seeding, planting, etc.) in relation to the beginning of the dormant season becomes important. Protection against dormant season flood should not be expected of late-planted materials.

(9) The principal function of a vegetative lining is to keep the fast moving water and transported coarse materials away from the soil surface. Therefore, the lengths, number per unit area, and physical qualities of the tops are, with few exceptions, the most important attributes of the lining.

(10) Once the soil surface beneath the tops has started to erode, exposing some of the roots, the lining has started to fail.

(11) Time is also an important element in the destruction of most streambank linings. It is usually best to think of the destructive process as an erosion, i.e., a little bit at a time, and at different rates for different conditions.

(12) The roots are also important. They should be sufficiently extensive to resist the pull of the tops resulting from the drag of the flow. Also, in case of partial failure, they help to retard the water speeds when exposed and, if not too severely damaged during a flood, may again support growth. Unlike vegetation generally, the roots of some willows and some of the other kinds of riparian vegetation, when once established, not only persist but thrive in water flowing at moderate speeds.

(13) Vegetative linings possess a healing ability under some conditions. The subsequent recovery of growth, following partial failure in an extreme flood, may induce sedimentation and bank buildup during the lesser floods that follow. The kind and quantity of the sediment load of the stream is undoubtedly a factor in this.

(14) The flow-retarding ability of the vegeta-

tion and thereby its ability to protect the soil surface is also dependent upon its toughness and resilience, things generally associated with growth and life. Aside from the vulnerable period during establishment, the critical periods are generally the dormant periods. If floods occur during the dormant season, it is desirable to select those types and varieties that retain much of their strength, resilience, and perhaps some life in the tops during this period. An excellent grass for the Northeast is creeping red fescue.

(15) Legumes are generally weak in retarding flow, and should be used only if they promote better growth of the other species in the mixture or have the ability to grow and persist where other materials do poorly.

(16) Low-growing woody materials, such as small types of willow, appear to have favorable qualities. If they can be induced to grow along the concave banks of streams, where protection is really needed, they act as skid surfaces for ice and transported debris. They must be pliable and capable of quick recovery from severe mauling. Brittle materials could offer little protection. Isolated, tough, bushy materials are undesirable because they are a major obstruction to the flow, causing high, destructive water speeds in their immediate vicinity. Early spring, when moisture and temperature conditions are favorable, has been found best for planting the woody vegetation.

(17) The mixture of woody and herbaceous materials should be such that the soil surface is protected, either by a very dense stand of shrubs or by shade-tolerant grass in a less dense stand of woody growth.

(18) Although the strength or resisting abilities of vegetative linings along streambanks have not been evaluated, there is some helpful information available about sod-type of growth in small waterways. From the data of Ree and Palmer (*7*), allowable values of shear stress for bermudagrass linings can be deduced.

Equivalent Stone Sizes for Bermudagrass and Woody Vegetative Linings

The notion of an equivalent stone size for describing the resistance of vegetation to destruc-

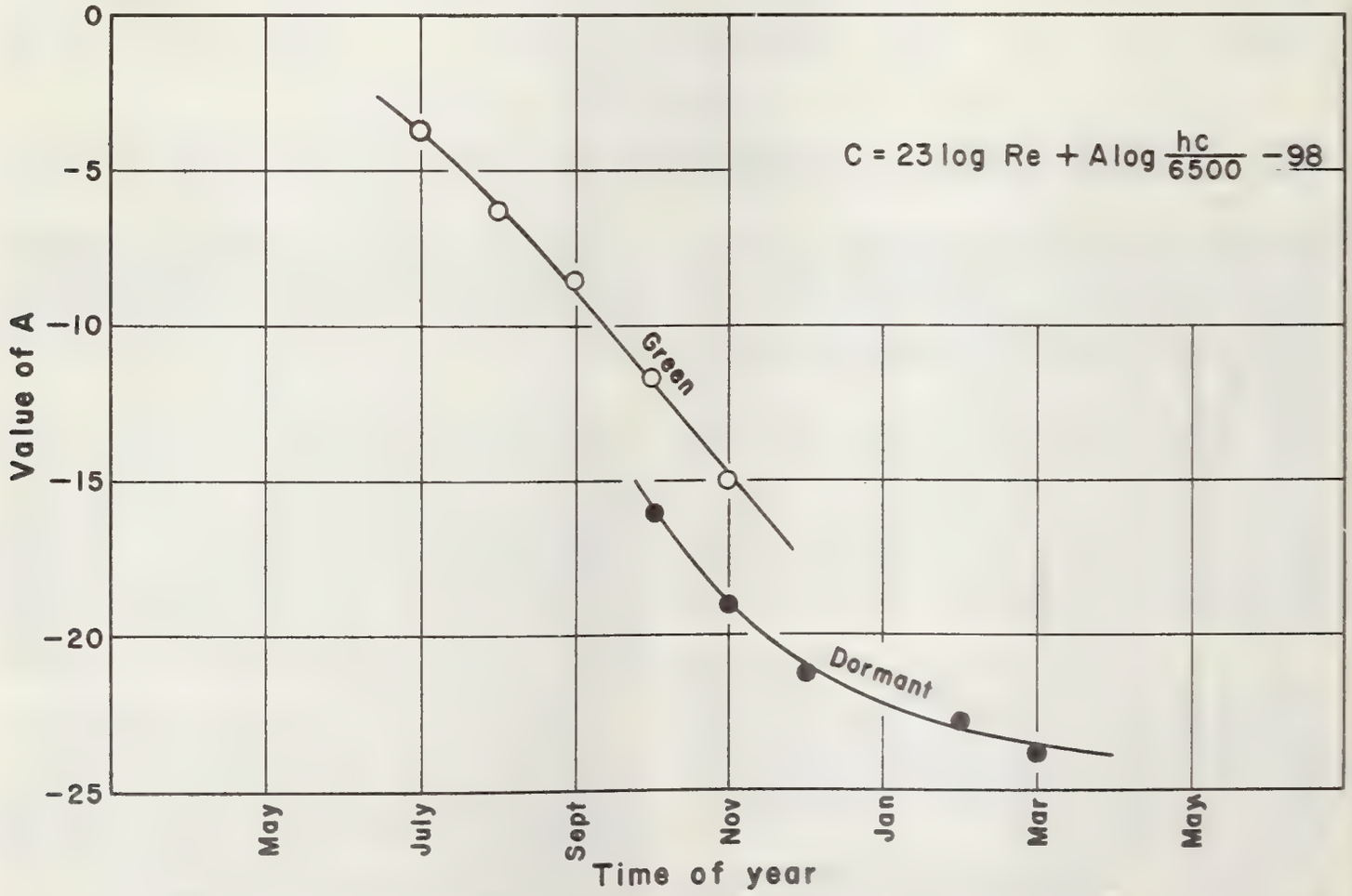

FIGURE 5. — Seasonal variations of A (Spartanburg bermudagrass channels).

tion by flowing water seems to have utility. Ree and Palmer (*7*) specifically give only allowable velocities for vegetated waterways, but their tabulated values of slopes and hydraulic radii for their many tests make it possible also to compute the allowable or limiting shear stresses and to derive the equivalent stone sizes given in table 1.

TABLE 1.—*Equivalent stone sizes for bermudagrass linings*

Condition of bermudagrass	Allowable shear stress	Equivalent stone diameter
	Lb./sq. ft.	*In.*
Fair stand, short,[1] dormant	0.9	2
Good stand, kept short, dormant	1.1	2
Good stand, long,[2] dormant	2.8	5.5
Excellent stand, kept short, green	2.7	5.5
Good stand, long, green	3.2	6.5

[1] Less than 5 inches high.
[2] Greater than 8 inches high.

The equivalent stone sizes of table 1 are based upon the relation that critical shear stress in pounds per square foot is equal to one-half the stone diameter in inches, the conversion factor being based upon tabulations of data from hydraulic laboratory tests and other sources by Chang (*1*), Lane (*3*), and others. The critical shear stress values shown are for Cecil sandy loam soil.

The existing information on the resistance of good sod during the dormant season indicates an equivalent stone size of from 2 to 5 inches, but there is practically no comparable information for woody materials. The single bit of evidence is for an excellent 3-year growth of basket willow and grass along Bennettsville Creek, New York (*4*). Here a stone size equivalent of a little less than 6 inches is indicated. Although this information is less than desirable for evaluating woody vegetative streambank linings, it does serve on a contingent basis to give the order of magnitude of the bank protection offered.

Some Practical Limitations

The role of vegetation in the regimen of streams varies with the size and slope of the stream, the frequency and duration of floods, the climate, the sterility or productivity of the soil materials along the stream edge and their inherent erodibility, the bed material texture and transport rates, land use, and presumably other things. The list of factors affecting the utility of vegetative materials for streambank protection are large; yet the range is not so large and complex that areas or situations cannot be delineated wherein certain kinds of vegetation are ideal for bank protection.

On the smaller streams of perhaps 100 feet or less in width and with moderate to low width/depth ratios, mature vegetation along the banks affects the average water speeds in floods and thereby the erosive agent. The transport capacity of the floodflows for bed material is also affected. Changes in this streambank growth by purposeful destruction, trimming, mowing, or spraying will affect the water velocities and thereby the channel capacity, the sediment transport capacity, the ability to scour the bed and the banks, and the meandering tendency. Utilization of these practices in timely maintenance can provide the most economical means for obtaining satisfactory channel conditions. Overdoing these things can result in a wildly meandering or severely degrading channel.

There are some cases on smaller streams where the effects of grazing and trampling by livestock along the edge of the stream are sufficient to induce objectionable streambank erosion. Close grazing and trampling reduce the protective capacity of vegetation. Fencing, along with the good fortune of having lower than normal floodflows for a few years, may be the least costly, effective corrective measure. However, experience has taught that annual fencing repair costs can be appreciable, and effective maintenance of fences for protection of streambank vegetation has been hard to obtain.

The very great importance of the studies of vegetative streambank linings, and the ascertainment of the portions of the bank where they are suitable, can be obtained by comparing costs. After shaping the bank, it costs about 20 cents per square yard of bank area to establish a herbaceous lining, 50 cents for a combination shrub and grass lining, and $4 or $5 for 12-inch-thick stone riprap. These costs are based upon experiences in western New York State in the protection of streambanks that were subject to floodwater erosion.

Many of the thoughts that have been expressed herein, along with others in regard to the use of vegetation along Buffalo Creek, New York, have been ably reported in a paper by Harry Porter and Leon Silberberger, entitled "The Use of Vegetation in Streambank Stabilization," given at the 1958 meeting of the Soil Science Society of America; also, "Streambank Stabilization" by the same authors (*5*). A recent article describing the use of vegetation in streambank protection is by Stanton and McCarlie (*8*).

Summary

Some factual information about the resistance of vegetation to destruction by floodflows

is given along with many observations and deductions about the role of vegetation in streambank erosion.

Little-publicized basic information, obtained by W. O. Ree, about the variation of water speeds within a flow of water over vegetation is again presented.

The retarding effect of grass on the flow of water as it varies with grass density, length, age, and time after frost is shown.

The protective ability of grass and woody vegetation in terms of equivalent stone size and the guiding principles for successful use of vegetation for streambank erosion control are given.

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SOME ASPECTS OF FLUVIAL MORPHOLOGY INFLUENCING INVESTIGATIONS OF CHANNEL STABILITY

[Paper No. 21]

By Elliott M. Flaxman, *sedimentation specialist, Soil Conservation Service*

Introduction

Proper identification, correlation, and sampling of representative soil materials facilitates the design of stable channels. Improvements may be made over many miles and intensive investigation can be costly, unless an interpretation of the fluvial morphology is possible. This interpretation is defined as a determination of the depositional environment in which the alluvium has accumulated to a depth at least as great as the proposed channel investigation. Stratigraphic correlation of deposits by depth and width across the valley would be part of the interpretation of fluvial history.

Interpretation of depositional environment is not only important for purposes of correlation but also because the mode of accumulation does much to establish the stability characteristics of the resulting deposit, if not immediately, then perhaps with time. Channel deposits are noncohesive because of the absence or scarcity of clay or fine silt that could act as a binder. On the other hand, flood plain deposition offers the greatest opportunity for a stable soil to develop, assuming relatively quiet water when fine material accumulation occurs. Fan sediment occupies an intermediate position from the standpoint of texture, since small particles are forced to deposit along with the coarse in the spreading flow.

This paper presents data available from channel and damsite investigations in alluvial valleys and on fans where the depositional environment is believed to be identifiable. An analysis of the data is made for the purpose of quantitatively establishing the distinction between channel, fan, and flood plain deposits. Problems associated with a quantitative distinction between such sediments are also discussed.

Data on Fluvial Sediments

Table 1 lists the available data from 32 stream valleys in 7 Western States. The amount of such data from each valley varies considerably. Nevertheless, the data are representative of specific valleys, if they are not necessarily a fair sample of all alluvial soils in the west. A major separation of the information is between channels, fans, and flood plains. The flood plains are further subdivided into sediments deposited in recent years and those of older but variable history. For the most part, they include sediments with profile development. They either occur in valleys known to have a very slow rate of accumulation because of low sediment yield or because of dissection of the flood plain by valley trenching.

Stream slopes were obtained in many cases from topographic sheets, some of which were of small scale and large contour interval. They should therefore be considered as approxima-

TABLE 1.—*Size distribution of fluvial sediment by depositional environment, Western States*

Depositional environment	No.	Stream	County	State	Stream slope	Size distribution				Plasticity index	Coefficient of sorting[1]	Rate of particle-size increase[2]
						25 pct. finer	Median	75 pct. finer	<.005 mm.			
					Ft./ft.	*Mm.*	*Mm.*	*Mm.*	*Pct.*			
Channels	1	Adobe Creek	Lake	California	0.003	0.90	4.00	12.00			3.67	0.222
	2	do	do	do	.003	1.50	5.20	10.00			2.58	.170
	3	do	do	do	.003	1.60	8.50	22.00			3.70	.408
	4	do	do	do	.003	3.50	15.00	27.00			2.78	.471
	5	Arroyo Grande	San Luis Obispo	do	.005	.75	2.80	7.00			3.06	.124
	6	do	do	do	.004	.36	7.00	15.50	2		6.54	.303
	7	Beaver Creek	Marion	Oregon	.0016	.20	4.00	40.00	7	11	14.10	.796
	8	do	do	do	.0016	.82	3.50	80.00			9.78	1.584
	9	Boise River	Canyon	Idaho	.0024	.14	.21	.27			1.39	.003
	10	do	Ada	do	.002	.48	.82	1.70			1.88	.025
	11	Calleguas Creek	Ventura	California	.0013	.10	.22	.31			1.76	.004
	12	do	do	do	0013	.29	.41	.43			1.28	.003
	13	Cebada Canyon	Santa Barbara	do	.007	.25	.35	.55			1.48	.006
	14	Corbett Canyon	San Luis Obispo	do	.015	.045	.16	.45	12		3.16	.008
	15	Cottonwood Gulch	Ada	Idaho	.023	.58	1.20	2.60			2.12	.041
	16	Elk Creek	Boise	do	.015	.80	1.80	3.40			1.85	.084
	17	Frye Creek	Graham	Arizona	.016	.48	1.30	3.20			2.58	.054
	18	do	do	do	.016	1.20	4.80	14.00			3.42	.256
	19	Gabbert Canyon	Ventura	California	.016	.70	1.00	2.00			1.69	.026
	20	Graveyard Wash	Graham	Arizona	.007	.19	.32	.50			1.62	.006
	21	do	do	do	.007	.42	1.00	2.70			2.54	.046
	22	Green River	King	Washington	.001	.10	.145	.18			1.34	.002
	23	do	do	do	.001	.15	.196	.25			1.29	.002
	24	Grimes Creek	Ventura	California	.014	.48	.80	1.35			1.68	.017
	25	La Posta Creek	San Diego	do	.010	.30	.43	.90			1.74	.012
	26	do	do	do	.009	.54	1.09	1.82			1.84	.026
	27	Limekiln Creek	Los Angeles	do	.021	.13	.72	6.00	5		6.80	.117
	28	do	do	do	.021	.55	1.80	5.00	2		3.12	.089
	29	Las Posas	Ventura	do	.006			1.30			1.44	.010
	30	Main Street Canyon	Riverside	do	.040	.78	1.60	3.90			2.21	.156
	31	Moore Creek	Boise	Idaho		.80	2.10	3.40			1.85	.048
	32	Simi Wash	Ventura	California	.008	.39	.55	.95			1.56	.011
	33	Skokomish River	Mason	Washington	.003	5.80	12.00	38.00			2.46	.648
	34	do	do	do	.004	7.40	24.50	94.00			3.56	1.721
	35	do	do	do	.002	4.80	11.30	28.00			2.42	.466
	36	Stockton Wash	Graham	Arizona	.004	.32	.50	.71			1.49	.008
	37	do	do	do	.004	.13	.39	1.49			2.91	.027
	38	Tar Springs Creek	San Luis Obispo	California	.015	.14	.21	.47	3		1.84	.007
	39	Upper Meadow Valley Wash	Lincoln	Nevada	.003	.028	.54	1.50			2.31	.029
	40	do	do	do	.0066	.38	2.80	13.30			5.90	.260
	41	do	do	do	.011	3.20	9.00	16.50			2.27	.270
	42	do	do	do	.011	2.60	13.00	20.30			2.78	.350
	43	Antelope Valley	Los Angeles	California	.114	.067	.21	.43	8		2.49	.007
	44	Deer-Day	San Bernardino	do	.050	.57	5.00	51.00			9.40	1.07
	45	do	do	do	.050	.57	4.00	20.00			5.90	.39
	46	do	do	do	.050	.90	7.50	32.00			5.90	.622
	47	do	do	do	.050	.70	8.50	38.00			7.70	.747
Fans	48	Etiwanda Creek	do	do	.047	.60	2.35	15.00			5.00	.288
	49	do	do	do	.047	.25	1.40	24.00			9.84	.474
	50	Grove Creek	Utah	Utah	.120	.035	[3].90	13.00	9	8	19.35	.259
	51	do	do	do	.120	.040	[3].75	14.00	8	5	18.65	.279
	52	do	do	do	.120	.026	[3].24	2.60	10	4	10.00	.052
	53	Adobe Creek	Lake	California	.0025	.034	.08	.18	8		7.29	.003
	54	do	do	do	.002	.03	.07	.18	3		7.74	.003
Flood plains—modern	55	Santa Rosa Creek	Sonoma	do	.003	.025	.06	.17	11		2.60	.003
	56	Upper Meadow Valley Wash	Lincoln	Nevada	.006	.050	.50	2.50	3		7.08	.049
	57	do	do	do	.006	.0065	.02	.08	10	11	3.56	.0015
	58	do	do	do	.006	.021	.04	.18	7		2.92	.0032
	59	do	do	do	.006	.05	.32	3.00	5		7.72	.059
	60	do	do	do	.003	.029	.06	.18	7		2.48	.003
	61	Adobe Creek	Lake	California	.003	.03	.07	.16	7	3	7.30	.003
	62	do	do	do	.003	.03	.11	.50	9	4	12.80	.009
	63	do	do	do	.002	.016	.03	.062	12	23	1.97	.0009
	64	do	do	do	.0025	.012	.039	.11	12	12	3.02	.0018
	65	Beaver Creek	Marion	Oregon	.0022	.02	.16	1.20	12	5	7.76	.0236
	66	do	do	do	.0022	.0075	.035	.13	21	10	4.16	.0024
	67	do	do	do	.0016	.008	.031	.066	22	14	2.88	.0012
	68	do	do	do	.0016	.0029	.023	.055	28	18	4.36	.0014
	69	Green River	King	Washington	.001	.015	.019	.24			4.00	.0045
Flood plains—recent to mature soil development	70	do	do	do	.0005	.009	.02	.035	17	6	6.20	.0005
	71	do	do	do	.0007	.018	.031	.055	8		1.39	.001
	72	La Posta Creek	San Diego	California	.010	.17	.24	.74			2.09	.011
	73	do	do	do	.009	.18	.52	1.05			2.41	.017
	74	do	do	do	.009	.10	.18	.40			2.00	.006
	75	Monroe Creek	Sevier	Utah	.06	.23	2.90	30.00			11.50	.595
	76	do	do	do	.06	.25	1.50	14.00			7.50	.274
	77	do	do	do	.06	3.60	8.50	24.00			2.60	.428
	78	Snohomish River	Snohomish	Washington	.001	.014	.045	.45	11		5.57	.009
	79	do	do	do	.001	.036	.085	.12	9		1.83	.007
	80	Sutherlin Creek	Douglas	Oregon	.004	.035	.071	.17			2.20	.0027
	81	do	do	do	.003	.028	.052	.11	10	4	1.98	.0003
	82	do	do	do	.003	.0065	.030	.29	23	37	6.69	.0028
	83	Upper Meadow Valley Wash	Lincoln	Nevada	.012	.02	.055	.31	16	14	3.95	.005
	84	do	do	do	.012	.035	.21	3.00	11	9	9.25	.06
	85	do	do	do	.012	.13	.40	.90	3		2.62	.015

[1] $\sqrt{Q_2/Q_1}$.

[2] $\frac{Q_2-Q_1}{50}$

[3] Size distribution not adjusted for 15-20 percent of samples over 6 inches.

tions of the gradients. A comparison of these with stream gradients in other parts of the country would no doubt show the western stream slopes to be substantially higher. This is believed to be one explanation for the relatively small amount of fines in the flood plain deposits shown on the table in the column giving the percentage of material less than 5 microns in size. This factor is also related to the generally low plasticity indices. The particle sizes of which 25, 50, and 75 percent are finer, by weight, completes the information from the classification of materials. Computed data on the table include Trask's Coefficient of Sorting,[1] which is obtained from the formula $\sqrt{Q_3/Q_1}$, where Q_1 (1st quartile) is the particle size of which 25 percent are finer (D_{25}) and Q_3 (3d quartile) is the size of which 75 percent are finer (D_{75}). The sorting coefficient approaches 1.0 as the two quartiles become more nearly equal.

The last column on table 1 is a value for rate of particle size increase, which is the quotient obtained from the formula $\frac{Q_3-Q_1}{50}$ where Q_3 and Q_1 are the same as above and the divisor is the percentage difference between the two quartiles.

Analysis of Data

Streamflow characteristics as related to settling velocities of particles were considered as a means of obtaining a numerical differentiation between environments of deposition by size distribution. Channelized flow conveys the coarsest particles available, but the degree of sorting varies, however, from place to place and from flow to flow, depending upon the magnitude and duration of the discharge, as well as the availability of material. There should, however, be a range of particle sizes that reflects the dominance of the velocity component as the transporting and depositing agent. Similarly, the characteristics of flood plains, where flows are usually elevated above the area of coarse material transport and where water viscosity and local turbulence are important, should dictate the limits of particle sizes carried and deposited.

The coefficient of sorting is indicative of the environment of deposition under the above-cited conditions, but in a general sense only. For example, in table 1, the average coefficient for channels is about 3.0, for fans 9.4 (with 2 of the 10 items looming very large in the average), and flood plains about 4.8. The sequence is in the anticipated order, but individual sample data of one group exceed the averages in others. This is to be expected, since, during the alluvial process, time for sorting prior to deposition is frequently insufficient, with fines being forced out of suspension with the coarse particles in velocities otherwise sufficiently strong for continued transport. Dispersion and loss of water are also reasons for unpredictable sorting patterns; but it is predictable that sorting will be poor on fans.

In searching for a more specific numerical indicator of the depositional environment in alluvial valleys, it was concluded that the average increase in particle size could serve to bracket the conditions under which sedimentation occurs. For example, a relatively large average increase in size for each 1 percent added between the 1st and 3d quartile should indicate a rather high velocity but variable flow during the depositional process. On the other hand, when there is a small average increase in size, the flow involved should have been relatively uniform rather than widely fluctuating in velocity and related flow conditions. A streamflow under high velocity but very irregular conditions would occur on a gravel wash or on a fan. Flow in a well-defined channel of uniform depth provides a definitive particle size and indicates general movement as bedload.

The most appropriate quartile to use in conjunction with average rate of size increase to segregate depositional environments was obtained by graphical means. Figure 1 is a plotting of the D_{75} and average rate of size increase. The nearly perfect correlation on log-log paper demonstrates the dominance of the D_{75} on the average rate of size increase. It also demonstrates that use of the coarser fraction does not offer a means of graphically segregating environments of deposition. Figure 2 is another plotting using the median, or D_{50}, with greater success in segregating channel, fan, or flood plain deposition. Figure 3, which shows the relationship between the D_{25} and the rate of particle size increase, makes a relatively clear-cut distinction between depositional environments.

The location of a few samples in an environment different from that given in the table can, at least in considerable measure, be explained. Classification by depositional evironment was made on the basis of topographic position. It does not follow, however, that such a position precludes the accumulation of sediment under the conditions of another streamflow regime. For example, the single flood plain sample shown within the channel envelope on figure 3 (No. 75) belongs with the high-velocity streams only by the broadest generalization. The sample was obtained from the streambank in an intermountain valley, Monroe Creek, Utah, where the slope is about 6 percent. It is logical to assume

[1] TRASK, P. D. STUDIES OF RECENT MARINE SEDIMENTS, CONDUCTED BY THE AMERICAN PETROLEUM INSTITUTE. Natl. Res. Council Bul. 89, pp. 60-67. 1932.

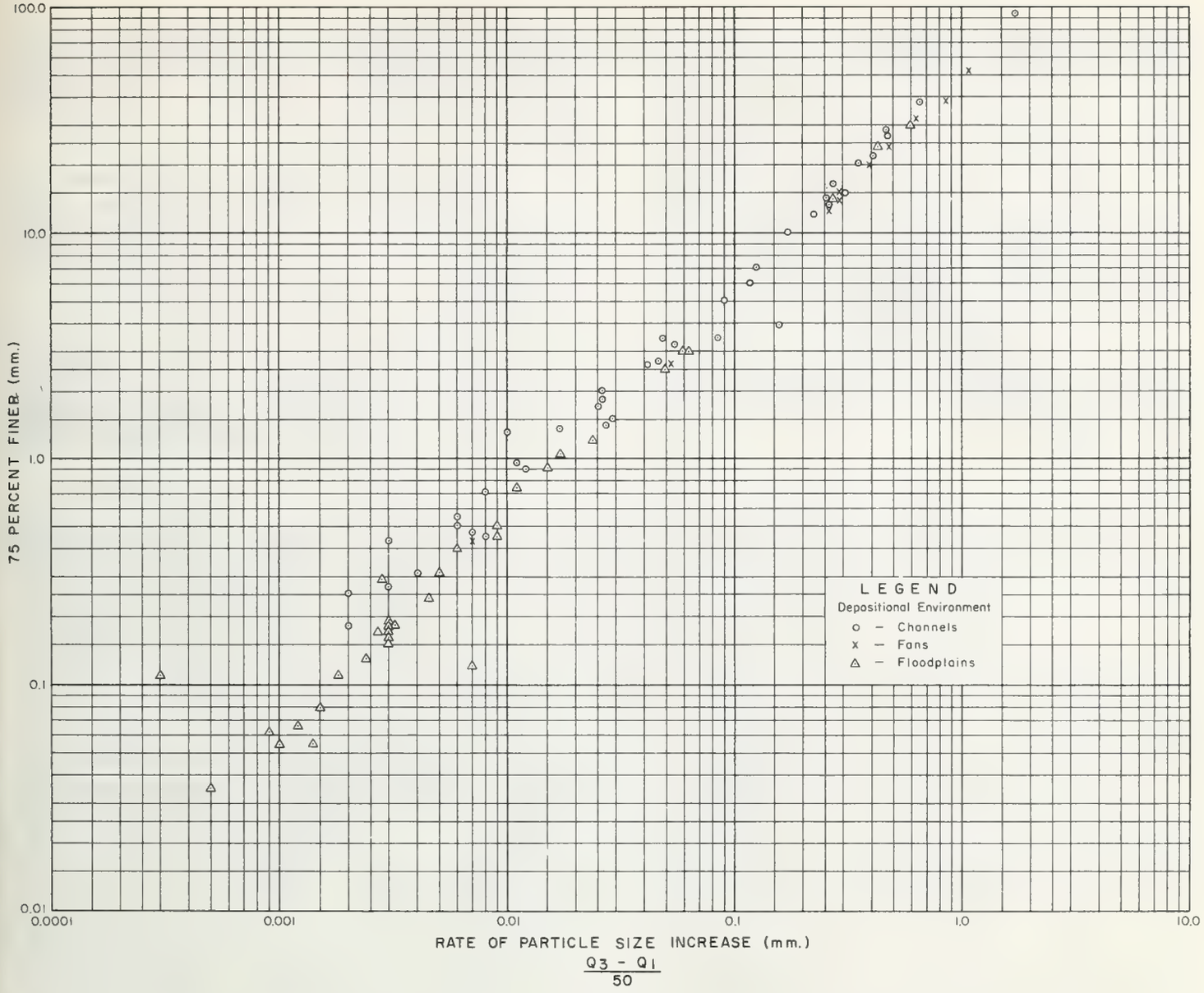

FIGURE 1. — Relationship of particle size increase to size of which 75 percent is finer.

that this deposit could have accumulated under channel flow conditions.

Similarly, one of the two channel events shown to be within the fan envelope occurs along a stream as it emerges from a mountain canyon. The site is located on Upper Meadow Valley Wash, Nev. The same is true for the channel sample (No. 27), in Limekiln Canyon, Calif., that is not within any of the envelopes. It is not unlikely that the sediment in each case accumulated under spreading fan conditions, although both samples were obtained from channels with insufficient capacity for even moderately large flood events. The other channel sample within the fan envelope on figure 3 (No. 6) is from Arroyo Grande channel in California. It was obtained near the mouth at the Pacific Ocean, where the presence of a beach barrier of littoral drift causes deposition of finer sediment at the D_{25} level than the average rate of particle increase of channel deposits would indicate.

One of the three samples identified as being from channels but which are plotted within the boundary of the flood plain envelope is from Stockton Wash, Ariz. (No. 37). This sample was obtained from a depth of 39 feet below the present channel. In this actively wandering stream, the flood plain deposits are certain to occupy varied positions with depth. An Upper Meadow Valley Wash sample identified as being from a channel (No. 39)), but is within the flood plain envelope, was obtained from near a place where the confined flow capacity is completely lost.

A sample from a fan deposit is located within

but near the boundary of the flood plain envelope. This (No. 52) and the two lowest D_{25} plottings within the fan envelope (Nos. 50 and 51) are from Grove Creek fan in Utah. As shown in footnote 3 of table 1, from 15 to 20 percent of the size distribution of these samples does not include materials over 6 inches in diameter. The Antelope Valley fan sample (No. 43) is derived from weathered shale on the slope above and may be properly classed as colluvium.

Post-Depositional Influences on Particle-Size Distribution

Changes in particle-size distribution can occur after deposition that may alter the indicated environment of accumulation. These changes include:

1. Deposition of finer or coarser sediment among the particles accumulating under a different flow regime. For example, voids between gravel depositing under a channel flow regime are filled with fine sediment during receding flows.

2. The process of eluviation or movement of fine particles downward in the profile. This process, over time and with sufficient rainfall, results in accumulations of clay size particles in a subsoil.

3. Weathering, or breaking down of particles by physical or chemical means.

4. Filling of voids by chemical precipitates such as calcium carbonate.

The first post-depositional process described is believed illustrated by samples Nos. 7 and 8

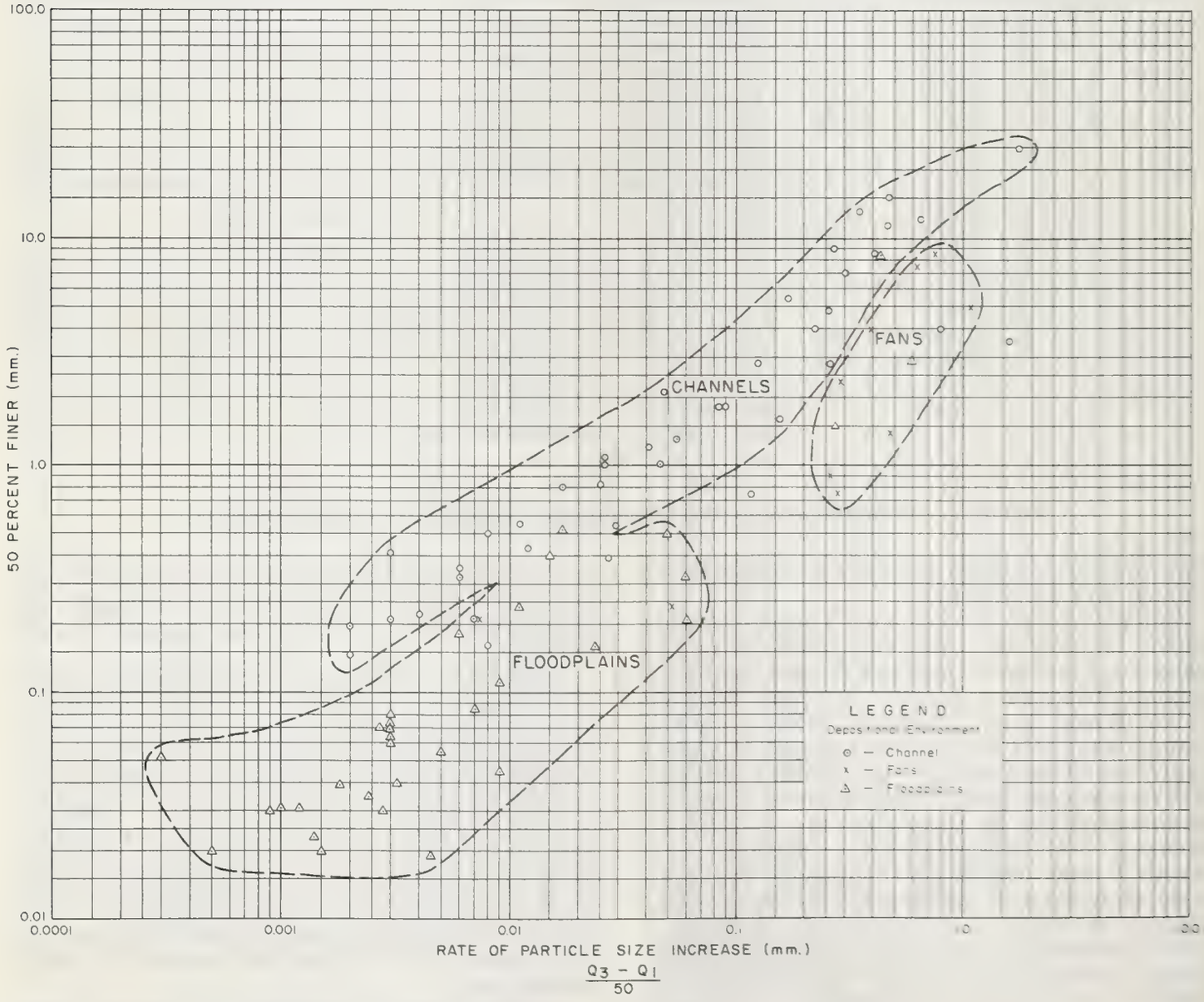

FIGURE 2. — Relationship of particle size increase to size of which 50 percent is finer.

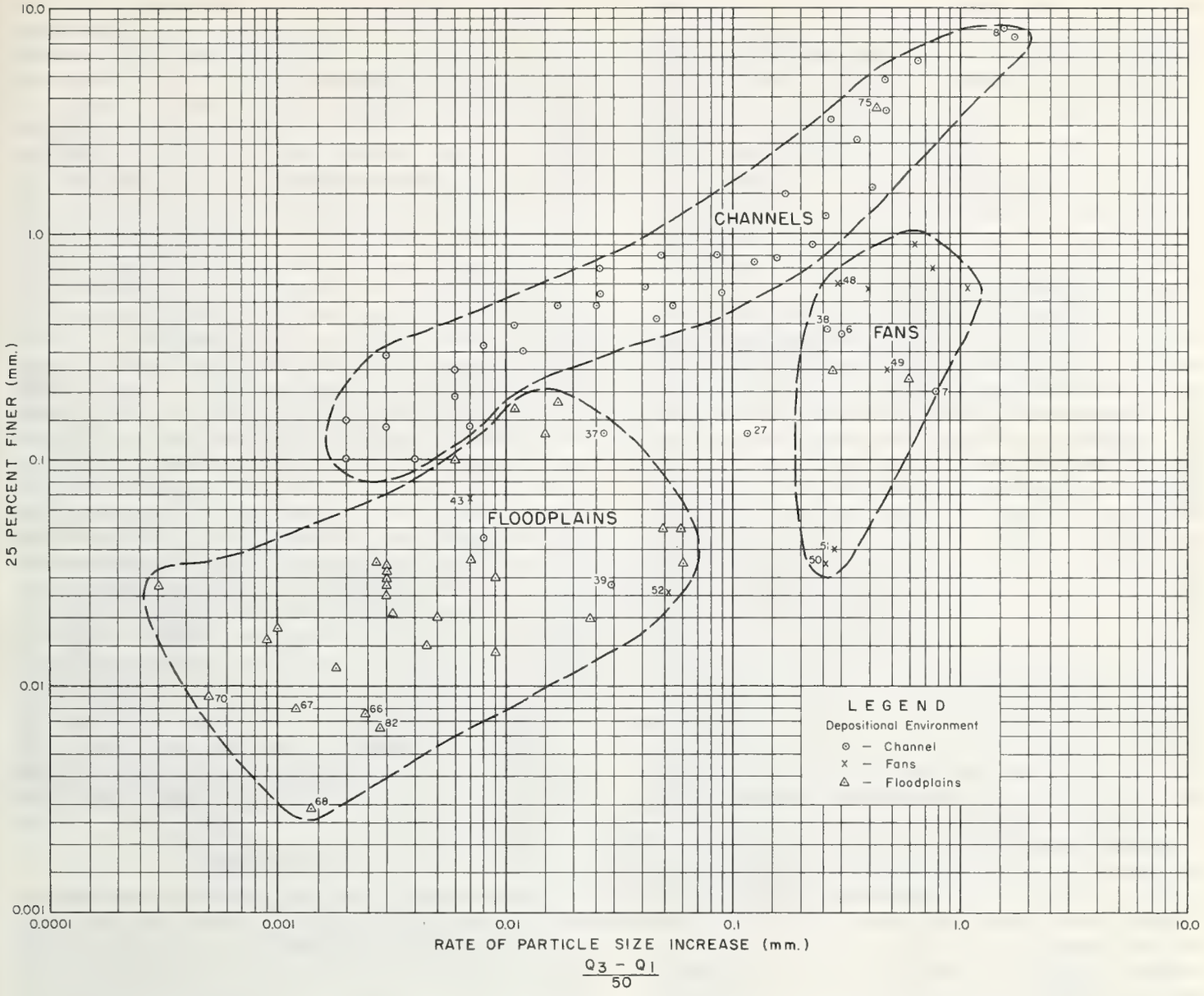

FIGURE 3. — Relationship of particle size increase to size of which 25 percent is finer.

on Beaver Creek, Oreg. The gravels in these samples were deposited by an ancestral Santiam River channel, which formerly occupied the lower Beaver Creek Valley. Fine sediment was subsequently deposited on top of the gravels. Perhaps because of a more protected site further up the creek valley, sample No. 7 had a considerable amount of fines deposited in the gravel voids. The mixing of sediment from two depositional environments places the plotted point within the confines of the fan envelope. Sample No. 8, obtained from the same gravel horizon near the mouth of the present creek, has a normal position in relation to other channel deposits.

Examples of a combination of the second and third post-depositional processes described are the older fine alluvium in Beaver and Sutherlin Creek Valleys in Oregon (Nos. 66, 67, 68, and 82) and Green River, Wash. (No. 70). The high proportion of fines relative to the rate of particle size increase is believed indicative of post-depositional eluviation and weathering. Age alone, however, does not necessarily result in alteration of deposits. The deposit at a depth of 50 to 60 feet on East Etiwanda fan in southern California is not significantly different from the one near the surface. This illustrates the slow rate of change in material characteristics in semiarid and arid climates.

Relation Between Depositional Environment Determinations and Channel Stability Investigations

It has been pointed out that stratigraphic correlation of beds whose environment of depo-

sition has been determined within an alluvial valley facilitates investigations pertaining to channel stability. It is now proposed to illustrate this by three examples from stream valleys listed on the table. These are, in increasing order of complexity, Skokomish River, Wash.; Adobe Creek, Calif.; and Green River, Wash. The early history of the Skokomish and Green River valleys reflects their proximity to the ocean base level. During a period of much lower sea level their valleys were eroded by glacier and stream action.

The Skokomish Valley was backfilled with gravel outwash to which has been added in more recent times, stream gravels and fine flood plain alluvium. The river moves back and forth across its relatively wide valley, cutting the banks on the outside on bends, rebuilding on opposite slip-off slopes. As shown by figure 4,*A*, the separation between gravel and fine alluvium is sharp and occurs at about the same elevation across the valley. This is a characteristic of gravel-floored valleys whose channels move by lateral erosion and accretion. Gravel outwash deposited on top of fine alluvium during flood flows drops back to the bedload platform as bank erosion engulfs the deposit. The fluvial characteristic of this valley will be maintained until a shift in base level occurs, due either to sea level change or earth movements. In this instance, the identification of the depositional environment of the two major sediment types and their correlation is straightforward.

Adobe Creek Valley, of which a part is shown on figure 4,*B*, is similar to Skokomish River Valley in some characteristics. That is, the stream moves laterally across the valley with the shifting gravel surface as the pivot. For several reasons, including lack of vegetative entrapment, the accretion is not proceeding as at Skokomish River Valley. About halfway through its course from foothills to Clear Lake, the stream enters a new environment, consisting of fine, relatively erosion-resistant sediment. This sediment may have accumulated at a time of higher lake level. In any event, the gravel flooring of the stream was introduced into the downstream reach by a conveyance channel cut into the fine alluvium. Borings made adjacent to it fail to reveal other channel-type deposits at the same level, indicating that natural changes in alinement are probably made by sudden evulsion rather than by lateral erosion. Item Nos. 1 to 4 in table 1 are the data on channel samples, and Nos. 53 and 54 are modern flood plain sediments. Item Nos. 61 to 64 are the old sediments in which the lower channel is being confined.

The Green River Valley has apparently had a chequered history during the postglacial epoch. Mullineaux [2] has described the geomorphic history of this valley and the adjacent and substantially larger White River, which in times past has joined the Green River in flowing to Puget Sound. The valley has backfilled with clean medium-sized sand overlain by discontinuous cross valley beds of silty sand, silt, clayey silt, and peat, at depths germane to present problems of stratigraphic correlation. A detailed analysis of a recent channel stability investigation has not yet been made. The research reported by Mullineaux and the character of the sediment obtained from borings indicate, however, that the valley was probably at one time nearly covered by a sandy wash with the channel shallow and wide. Depressions near the valley boundaries provided a suitable environment for lush vegetative growth from which peat subsequently formed. This environment was followed by or overlapped one favorable to the accumulation of fine flood plain deposits; then lateral erosion in a highly meandering channel dissected these sediments at various places and replaced them with fine sands, silty sands, and silts. The old meander pattern is still visible on aerial photographs. Natural levee deposits, such as exist along the banks of the present channel, may have contributed to the discontinuity of the bedding. Figure 4,*C* is a schematic representation of the described features in the Green River Valley.

Conclusions

A method has been described for differentiating between channel, fan, or flood plain deposits in the alluvial valleys from which data are available. The procedure involves a determination of the rate of particle size increase and plotting this value in relation to the size of which 25 percent is finer. The 25-percent size proved to be the best of the three levels of size distribution tested.

The described procedure is believed to be most helpful in segregating environments when deposition could have occurred either from transport as suspended material or as bedload.

It was pointed out that the correct interpretation of the fluvial morphology based on size distribution must consider postdepositional influences and that these influences can be reflected in differences in behavior and in deviations from the average position on the graph.

Finally, the fluvial morphology of three stream valleys was briefly discussed, in order to demonstrate the relation between environments of deposition and the stratigraphy of alluvial valleys. A balanced study of all the factors is important in the planning and design of channel improvements.

[2] MULLINEAUX, D. R. GEOLOGY OF THE RENTON, AUBURN AND BLACK DIAMOND QUADRANGLE, WASHINGTON. U.S. Geol. Survey Open File Rpt., 202 pp. 1961.

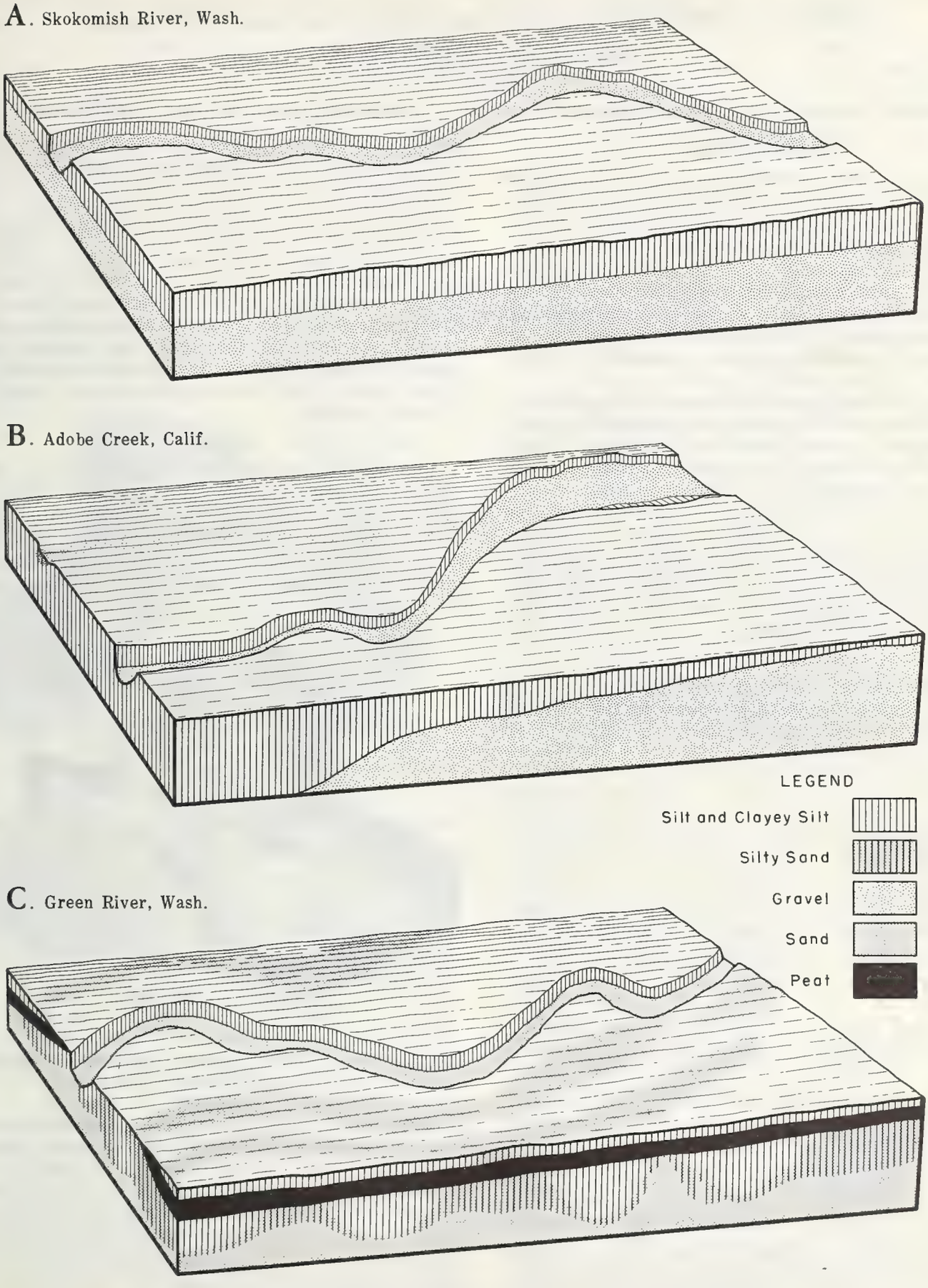

FIGURE 4. — Stratigraphy of alluvial deposits in three western streams.

SUSPENDED SEDIMENT CONCENTRATIONS IN A MICHIGAN TROUT STREAM AS RELATED TO WATERSHED CHARACTERISTICS

[Paper No. 22]

By W. DAVID STRIFFLER, *research forester, Lake States Forest Experiment Station, Forest Service*[1]

Introduction

The northern forested region of the Lake States has long been noted for the many fine quality streams and lakes that originate there. The quality of the water in these streams has generally remained high as long as the natural stream channels have not been disturbed. However, few streams have escaped disturbance in some manner. The net result has been to increase the sediment load carried by these streams.

The disadvantages of sediment in the streams are well known. However, in northern Michigan streams, one of the chief causes for alarm is the effect upon the trout resource. This effect is well documented (*2*, *4*).

The purpose of the study reported here was to examine the suspended sediment concentration in a representative northern Michigan watershed and evaluate the sediment load with respect to geology, soils, land use, and other watershed characteristics.

The Area

The watershed selected for this study is the Tobacco River watershed located in the central part of Michigan's lower peninsula (fig. 1). It includes areas representative of both the northern forested region and the southern agricultural region of the State. Therefore, the wide variety of geological formations, soils, and land-use types offers a broad range of sediment-producing conditions. The river basin is divided into three main branches, all of which head up in the forested upper watershed and then flow through the agricultural lower watershed (fig. 1). The waters of the river are largely considered marginal trout producers, although the upper parts of all three branches are classified as trout waters.

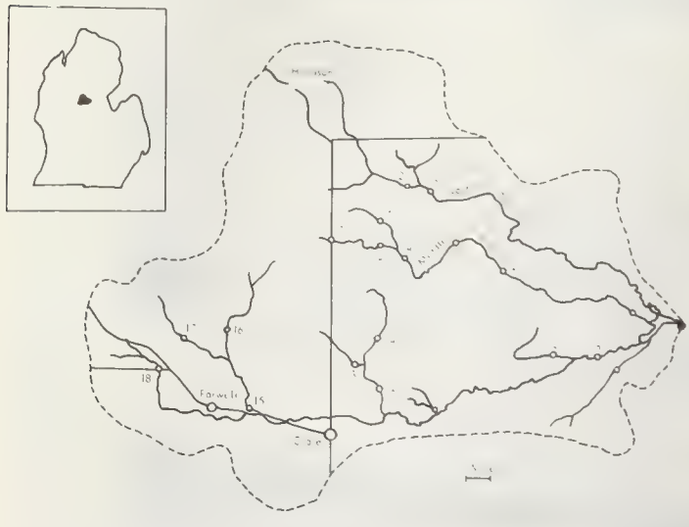

FIGURE 1. — Sampling stations on 20 small watersheds within the Tobacco River basin. Location of the basin in Lower Michigan is shown in the upper left corner.

Geology

The entire watershed is covered by a deep mantle of glacial drift. Four major types of surface deposits are commonly recognized. They include moraine, outwash plains, till plains, and glacial lake bed plain (fig. 2). The main streams flow through each of these from west to east.

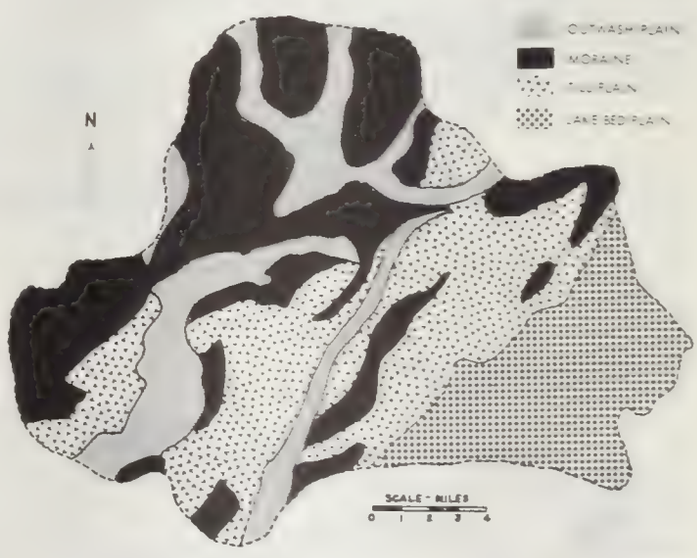

FIGURE 2. — Surface geology, Tobacco River Watershed. (After Martin (*9*)).

Within the morainal headwaters on the west, the landscape is hilly to rolling, soils are sands or sandy loams, and streams are small and widely separated, and they have a uniform flow. Intermingled outwash plains consist of ancient glacial drainage channels where soils are largely stratified sands and gravels. Such deposits are generally level and frequently form channels for present-day streams.

[1] This study was conducted in cooperation with the Michigan Water Resources Commission, the Michigan Conservation Department, and the University of Michigan.

The till plain deposits occur within the central part of the watershed. They vary from level to rolling and are composed of heavy clays and clay loams. Streams are more finely branched than in moraine or outwash areas and exhibit a wide range of flows during the year.

Lakebed plains occur in the eastern end of the watershed. They are broad level plains composed of sand but underlain by lacustrine clay at varying depths. Surface drainage is almost nonexistent except for artificial drains and the main branches of the Tobacco River.

Soils

Although no detailed soil surveys of the watershed area exist, a generalized map of major soil associations within the watershed is available (*10*). Primary soil mapping units (fig. 3) include six soil associations. Soil associations are closely related to the geologic deposits from which they develop.

FIGURE 3. — Major soil associations, Tobacco River Watershed. (After Veatch (*10*)).

SA– 7–*Nester-Iosco-Emmet.* This is primarily a till-plain derived soil consisting of a friable, bouldery till overlain by a thick cover of eluvium ranging from clay loam to sandy loam. Topography is rolling with numerous basin depressions.

SA–16–*Emmet-Roselawn.* These are upland morainal soils that vary in their surface composition according to the nature of the underlying drift, although sandy loams and loams predominate. Topography is rolling to hilly with irregular drainage features and numerous basin depressions.

SA–22–*Roselawn-Emmet-Rubicon.* These soils are also morainal soils and differ from SA-16 only in that surface soils are sands rather than loams and sandy loams. This association occupies 29 percent of the Tobacco River watershed.

SA–24–*Grayling-Rubicon.* These are the major soil types developed on the deep sandy outwash plains. Soils are medium to coarse sands with varying degrees of profile development. Relief is generally flat.

SA–35–*Muniscong - Ogemaw - Bergland.* These are lake plain soils and characteristically consist of sand and gravel deposits underlain by clay. Surface soil may include areas of dry sands or clays, but wet sands predominate.

SA–48–*Ogemaw-Rubicon-Nester.* This association occurs on both till plain and outwash plain deposits. The distinguishing characteristic of these soils is a clay subsoil overlain by sand and gravel. The clay may lie at shallow depth or may come to the surface. The topography ranges from level to undulating with occasional basin depressions.

Land Use

Land use within the watershed is closely related to the surface geology and soils. Moraines with deep sandy profiles and sandy surface soils are largely forested. Till plains are largely agricultural with use divided between cultivation and pasture land. Outwash plains are divided between pasture and forest. The use of lake plains depends upon the depth of the sand mantle and the drainage within a particular area. Where the clay beds come to the surface, cultivation predominates, but where the sand mantle is deep or surface soils very wet, forest and pasture lands predominate. For purposes of this study, land use was classified into four types:

Cultivation.—Land area devoted to cultivated crops. Primary crops include corn, small grains, and other crops requiring annual tillage.

Pasture land.—The land area devoted to sheep or cattle pasture. Hay lands were also included in this category.

Wild land.—Included abandoned farmland, brushland, or other predominantly open land not devoted to any specific use.

Forest land.—Land area supporting a forest cover. Major forest types in the watershed included northern hardwoods, lowland hardwoods, oak, aspen, and small areas of pine plantations.

Methods

The primary objective was to relate the suspended sediment concentration in the stream to streamflow, land use, soils, geology, and the other watershed characteristics measured, by

means of a multiple regression analysis. To accomplish this, 20 sample watersheds within the Tobacco River basin were selected, each representing different conditions with respect to size and watershed characteristics. Suspended sediment samples and stream discharge measurements were collected at irregular intervals between September 1960 and April 1962. Hydrologic events occurring during the sampling period included two snowmelt periods and several major rainstorms with return intervals of about 5 years. A total of 840 sediment measurements were made, about half of them during snowmelt or stormflow periods.

Several notable studies have used techniques similar to this (*1, 2, 3, 5, 6, 7*). The one major difference in this study is the form of the dependent sediment variable. All the studies cited used some expression of sediment yield over a period of time, while in this study the individual observations of sediment concentration or sediment discharge rates form the dependent variable.

The 21 variables used in the analysis are summarized in table 1. Sediment concentrations represent the average stream sediment concentrations at the time of sampling. Sediment samples were collected with the US DH–48, US DH–59,[2] and automatic single-stage samplers. Sediment discharge rates were computed from sediment concentration values. Stream discharge rates were determined from stage-discharge curves prepared for each sample watershed. Watershed areas were measured from aerial photographs. Channel lengths were measured on a large scale (1:22,240) base map of the watershed. Stream channel gradients were determined either from USGS topographic quadrangle sheets (scale 1:62,500, 10-foot contour intervals) or from direct measurement with a precision surveying altimeter. The number of road crossings was also determined from the large-scale base map. The length of eroding banks was determined from a stream survey and plans report by the Fish Division, Michigan Conservation Department. The proportion of each sample watershed classified in the various geologic types and soil associations was measured directly from the soil and geology maps (figs. 2 and 3). Land use classification was based on aerial photographs and a field survey. Values of these variables for the individual sample watershed are presented in table 2. Various combinations of these variables were also included in the analysis.

TABLE 1.—*Summary of watershed variables used in the analysis of 20 small watersheds*

Symbol	Definition	Units	Range	Mean
Sed p.p.m.	Sediment concentration	P.p.m.	1–3800	115
Sed Q	Sediment delivery rate	Lb./sq. mi./day	0.1–157,800	1,340
Q	Stream discharge rate	C.f.s./sq. mi.	.001–13.168	1.641
R/F	Rising/Falling stage	Rising = 1; falling = 2		1.645
A	Area of watershed	Sq. mi.	1.0–134.3	21.8
CL	Length of permanent channels	Miles	0.3–34.9	6.6
CG	Gradient of main channel	Feet/mile	5–61	23.4
EB	Total length of eroding banks	100's of feet	0–62	10
RC	Road crossings	Number	0–83	10.7
Mor	Moraine	Percent	0–94	50
Out	Outwash	do	0–42	18
Till	Till plain	do	0–96	26
Lake	Lake plain	do	0–100	6
SA–7	Nester–Iosco–Emmet Soil Association	do	0–100	19
SA–16	Emmet–Roselawn Soil Association	do	0–50	7
SA–22	Roselawn–Emmet–Rubicon Soil Association	do	0–100	44
SA–24	Grayling–Rubicon Soil Association	do	0–44	18
SA–35	Muniscong–Ogemaw–Bergland Soil Association	do	0–100	6
SA–48	Ogemaw–Rubicon–Nester Soil Association	do	0–58	6
Cult	Cultivated land	do	0–45	10
Past	Pasture land	do	0–42	18
Wild	Wild land	do	6–37	26
For	Forest land	do	9–68	44

The multiple regression analysis was computed on the University of Michigan IBM 709/7090 computer, with a standard program known as the Westervelt Modified General Motors Multiple Regression Program. This is a stepwise analysis that selects and analyzes the most important variables among the data. A multiple regression coefficient, coefficient of determination, and *F* level are computed for each variable entering the analysis. In this manner, only the most important variables are included in the analysis while nonsignificant variables are dropped.

[2] Two sediment samplers developed by the Inter-Agency Sedimentation Project, St. Anthony Falls, Minn. *See* paper No. 25.

Results

In the multiple regression analysis, all the 23 variables listed in table 1 plus 7 interaction forms of these variables were tested against sediment concentration (p.p.m.) and sediment discharge rates (lb./day/sq.mi.). In the test against sediment discharge rates, they were found to be significantly related to 6 of the 28 variables. These six, accounting for 75 percent of the variation (equation 2, table 3), included stream discharge (Q), whether the stream was rising or falling (R/F), eroding banks (EB), and three of the soil associations (SA–22, SA–24, SA–48). In the test against sediment concentration (p.p.m.), the same six variables as above, plus the land use variable, cultivation, were significantly related. Fifty percent of the variation was explained by the seven variables (equation 1, table 3). These two equations permit the individual effects of the included variables to be evaluated. Fourteen variables remain that, since they are not included in the analysis, must either have no significant relation with stream sediments or else the relation is obscured by the variables included.

To determine the effects of these remaining variables, only selected variables were used in a series of analyses. In this manner the specific effects of the geologic types, soils groups, and land use types were evaluated. Regression equations derived from these analyses are also included in table 3. Only statistically significant variables are included in the equations.

Discussion of Significant Variables

Streamflow Variables

A large proportion of the total variation was explained by the two streamflow variables, discharge (Q) and whether the stream was rising or falling (R/F). When sediment concentration (*Sed p.p.m.*) was used as the dependent variable, 43 percent of the variation was explained by rising/falling stage. Less than 1 percent was explained by stream discharge (Q). When sediment discharge rates were used as the dependent variable, 52 percent of the variation was explained by stream discharge, while 20 percent of the variation was explained by rising/falling stage. These values indicate that, in northern Michigan trout streams, sediment concentrations greatly increase on the rising stage and that sediment discharge rates increase with increasing streamflow rates. This relation is illustrated in figure 4.

The strong relation between sediment and streamflow variables suggests stream discharge may be an important eroding agent as well as the vehicle of transportation. The consistently high significance of eroding banks in the analy-

TABLE 2.—*Summary of watershed characteristics for sample watersheds*

Station	Area	Total channel	Channel gradient	Road crossings	Eroding banks	Surface geology				Soil association (SA)						Land Use			
						Moraine	Outwash	Till	Lake	7	16	22	24	35	48	Cultivated	Pasture	Wild	Forest
	Sq. mi.	*Miles*	*Ft./mi.*	*No.*	*100 ft.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	36.6	37.5	11	23	38	40	17	36	8	43	2	26	13	9	7	11	26	28	34
2	27.4	23.6	14	19	28	49	21	30	0	40	3	36	18	0	3	10	26	28	34
3	22.8	16.8	13	13	15	59	25	16	0	30	3	45	22	0	0	7	23	32	37
4	15.2	7.5	12	7	0	73	24	2	0	7	4	58	31	0	0	3	12	36	48
5	8.3	3.9	15	4	0	70	28	2	0	0	0	62	38	0	0	4	11	37	47
6	4.7	1.7	19	0	0	90	10	0	0	0	14	57	29	0	0	1	3	37	57
7	1.0	.3	17	0	0	92	8	0	0	0	0	56	44	0	0	0	0	35	65
8	134.3	93.0	17	83	62	22	18	48	12	28	5	31	22	11	3	13	23	26	35
9	8.1	8.2	5	5	0	6	0	0	100	0	0	0	0	100	0	7	18	30	44
10	2.3	3.7	14	2	0	0	0	3	97	11	0	0	0	89	0	22	28	24	24
11	3.4	5.7	29	5	10	17	0	83	0	100	0	0	0	0	0	45	37	6	9
12	16.3	13.0	18	14	20	7	16	77	0	40	0	16	22	0	22	23	38	22	14
13	4.8	2.7	25	3	0	3	1	96	0	14	0	28	0	0	58	22	42	16	18
14	5.8	3.6	20	5	0	29	14	57	0	52	0	22	20	0	6	26	33	22	17
15	28.6	16.7	33	8	1	42	42	16	0	0	8	66	26	0	0	2	8	26	62
16	20.6	7.7	53	3	0	54	35	11	0	0	7	70	23	0	0	1	5	25	68
17	3.7	2.3	61	0	0	948	0	6	0	0	0	100	0	0	0	0	2	30	67
18	5.1	4.7	28	1	0	92	6	2	0	0	50	50	0	0	0	3	19	23	50
19	48.1	21.4	27	11	27	67	32	1	0	0	14	57	22	0	7	2	8	18	68
20	38.9	14.7	28	8	0	64	36	0	0	0	19	53	28	0	0	2	6	19	67

TABLE 3.—*Summary of multiple regression equations*

Equation No.	Equation	Correlations
1.....	$\log SED_{ppm}=3.066+0.191 \log Q+0.135 \log EB-0.003\ SA/22-0.007\ SA/24-0.003\ SA-48+0.001\ Cult-0.765\ R/F.$	$r=0.72,\ S_y=0.420$
2.....	$\log SED_Q=3.831+1.190 \log Q+0.134 \log EB-0.003\ SA/22-0.007\ SA/24-0.003\ SA-48-0.764\ R/F.$	$r=0.87,\ S_y=0.419$
3.....	$\log SED_{ppm}=2.996+0.131 \log Q+0.154 \log EB-0.767\ R/F-0.002\ (Mor+Out).$	$r=0.70,\ S_y=0.426$
4.....	$\log SED_Q=3.729+1.128 \log Q+0.153 \log EB-0.780\ R/F-0.002\ (Mor+Out).$	$r=0.87,\ S_y=0.425$
5.....	$\log SED_{ppm}=3.038+0.141 \log Q+0.134 \log EB-0.778\ R/F-0.003\ (SA-16+SA-22+SA-24).$	$r=0.71,\ S_y=0.425$
6.....	$\log SED_Q=3.771+1.138 \log Q+0.131 \log EB-0.778\ R/F-0.003\ (SA-16+SA-22+SA-24).$	$r=0.87,\ S_y=0.424$
7.....	$\log SED_{ppm}=1.421+0.011\ Past.$	$r=0.24,\ S_y=0.528$
8.....	$\log SED_Q=2.244+.005\ Past.$	$r=0.08,\ S_y=0.848$
9.....	$\log SED_{ppm}=2.027-0.006\ (Wild+For).$	$r=0.23,\ S_y=0.528$
10.....	$\log SED_Q=2.470-0.003\ For.$	$r=0.07,\ S_y=0.849$
11.....	$\log SED_{ppm}=2.897+0.138 \log Q+0.093 \log CL-0.002\ (Mor+Out)-0.781\ R/F+.003 \log EB\times(Cult+Past).$	$r=0.71,\ S_y=0.424$
12.....	$\log SED_Q=3.630+1.134 \log Q+0.093 \log CL-0.002\ (Mor+Out)-0.781\ R/F+0.003 \log EB\times(Cult+Past).$	$r=0.87,\ S_y=0.424$
13.....	$\log SED_{ppm}=1.409+0.007\ Past+0.001 \log EB\times(Mor+Out)+0.002 \log EB\times(Cult+Past).$	$r=0.28,\ S_y=0.575$
14.....	$\log SED_Q=1.371-0.212 \log CL+0.325 \log EB+0.009\ (Mor+Out)+0.021\ Past.$	$r=0.30,\ S_y=0.813$

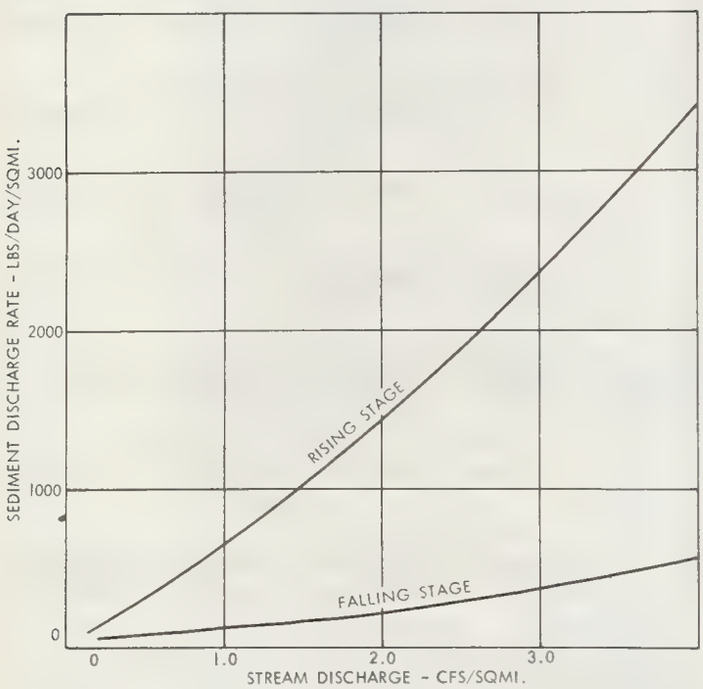

FIGURE 4. — General relation between stream discharge and sediment discharge rate for northern Michigan trout streams.

sis supports this conclusion. As the stream rises, after a rainstorm or snowmelt period, an increasing area of streambanks becomes subject to the stream action. Sediment immediately available to the stream includes loose soil material that has accumulated at the foot of the bank since the last storm flow and sheet erosion directly from the face of the bank during the storm. Additional sediment becomes available as the stream rises and begins to cut into the bank. All these processes tend to cause high sediment loads on the rising stage and at higher discharge rates. However, as the stream recedes, the decreased energy in the stream is no longer sufficient to cause bank cutting and, since all loose soil material has already been removed, that part of the sediment load derived from the channel is greatly reduced.

Channel Length

The variable, channel length (*CL*), representing the total length of all permanent stream channels within each sample watershed, was significantly related to sediment concentration and sediment discharge rate in equations 11 and 12. This raises the question as to whether watershed area is actually the value being considered, since channel length and area are closely correlated. However, since the area variable was included in these problems and rejected as nonsignificant, some other explanation is indicated. Actually, there are two reasons that channel length becomes significant. First, total channel length is related to the length of eroding banks within a watershed. It is reasonable to expect that the greater the length of the permanent channels within a watershed, the greater the length of eroding banks. Therefore, the inclusion of channel length may be a reflection of the high significance of eroding banks. Second, as the density of channels within a watershed increases, there is a greater opportunity for sediment derived from sheet erosion to reach the channels. The inclusion of channel length as a significant variable might also be due to this relationship. This suggests that a drainage density value for each watershed, as suggested by Horton (*8*), might be a much better variable to use than channel length.

Geology

If equations 3 and 4 are used, the contribution of sediment from the combined moraine

and outwash geologic types can be evaluated. By differences, the contribution of the remaining land area, till plain and lake plain, can also be evaluated. The sources of sediment derived from the land surface, excluding sediment derived from eroding banks, is (1) 53 percent from moraine and outwash (68 percent of area); and (2) 47 percent from till and lake plain (32 percent of area). The range of average sediment discharge rates that might be expected from a theoretical "average" watershed with 100 percent moraine plus outwash is 442 lb./day/sq. mi. as compared to 700 lb./day/sq. mi. from the same watershed with 100 percent till plain.

Soils

Soil associations may also be evaluated with respect to sediment contribution (equations 5 and 6). In this problem, the soil associations were combined into similar type groups. *SA*–16 + *SA*–22 + *SA*–24 formed the group of sandy soils, whereas *SA*–7 + *SA*–48 formed a group of heavier textured loam and clay soils. *SA*–35, the lake plain soils, remained separate. Sediment contribution from the various soil groups is as follows: (1) 48 percent from *SA*–16 + *SA*–22 + *SA*–24 (69 percent of the area); and (2) 52 percent from *SA*–7 + *SA*–48 + *SA*–35 (31 percent of the area). These values represent erosion from the land surface and do not include sediment originating from streambank erosion. The range of average sediment discharge rates that might be expected from the "average" watershed with 100 percent *SA*–16 + *SA*–22 + *SA*–24 would be 369 lb./day/sq. mi., and the same watershed with 100 percent *SA*–7 + *SA*–48 would yield 736 lb./day/sq. mi.

Land Use

The effect of the various land use types upon the amount of suspended sediment in the streams can also be evaluated (equations 1, 7, 8, 9, 10). The proportions of suspended sediment originating from upland erosion and to be attributed to the use types are: (1) 13 percent from cultivated land (10 percent of area); (2) 42 percent from pasture land (18 percent of area); (3) 17 percent from wild land (28 percent of area); and (4) 28 percent from forest land (44 percent of area).

In this analysis, the proportion of sediment derived from pasture land is much greater than from cultivated land. This is probably due to differences in topography and length of overland flow to the stream. In the Tobacco River watershed, the cultivated areas are primarily confined to the level or gently sloping upland till plain areas, whereas much of the land along stream bottoms is pastured. Streambottom lands characteristically have rough topography with short steep slopes, and discharge surface runoff directly into the stream.

Land use effects are illustrated in figure 5. This shows the relation between land use type and sediment as it occurs. Since land use is so closely related to soil type and topography, this relation merely points out the important sediment contributors and cannot be used to predict the results of land use changes.

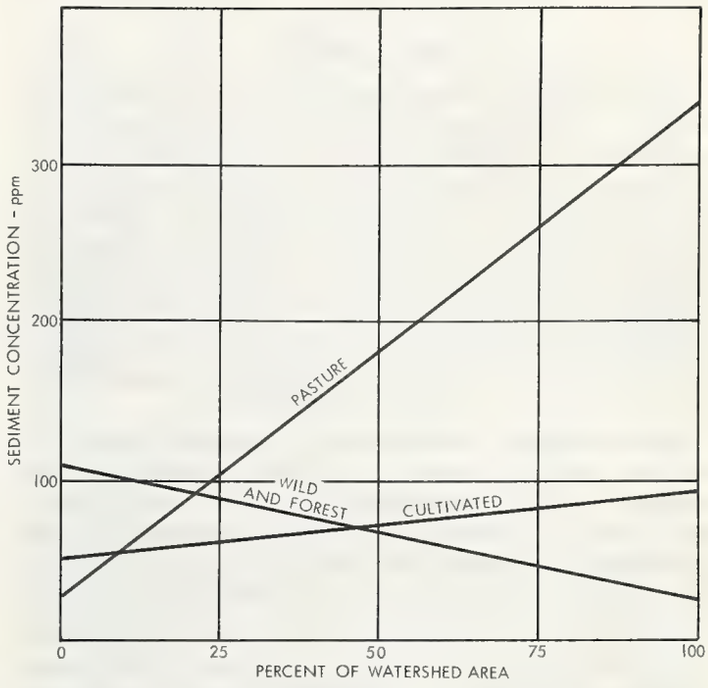

FIGURE 5. — General relation between land use and sediment concentration in the Tobacco River Watershed.

Eroding Banks

One of the more important sources of suspended sediment in the watershed is streambank erosion. Estimates of its contribution range from 26 to 30 percent of the average sediment load with an average of 28 percent (equations 1 to 6).

Erosion from streambanks represents one source of suspended sediment that can be effectively controlled in northern Michigan trout streams. To determine potential reductions within those sample watersheds containing eroding banks (watersheds 1, 2, 3, 8, 11, 12, 19), the values of the watershed characteristics for each sample watershed (table 2) were inserted into equations 3 and 4 and average sediment values computed both with and without the variable for eroding banks. Potential reductions in the average sediment load ranged from 30 to 47 percent (table 4). Whether such reductions can actually be attained remains to be tested.

TABLE 4.—*Predicted sediment reductions by stabilizing eroding banks*

Sample watershed No.	Eroding banks	Average sediment		Predicted sediment		Potential reduction
	100 feet	*P.p.m.*	*Lb./day sq. mi.*	*P.p.m.*	*Lb./day sq. mi.*	*Percent*
1.....	38	91	677	52	388	43
2.....	28	74	803	44	482	40
3.....	15	58	612	38	405	34
8.....	62	86	573	46	305	47
11.....	10	67	589	47	477	30
12.....	20	89	542	56	342	37
19.....	27	89	544	54	328	39

Conclusions

Several conclusions may be drawn from this study. First, the study demonstrates that watershed characteristics can successfully be evaluated with respect to their influence upon stream sediment concentrations or sediment discharge rates by using individual suspended sediment observations from the sample watersheds rather than total sediment yields as the dependent variables. An added advantage is that a large number of comparisons is possible for any group of sample watersheds.

In northern Michigan, a broad range of surface geology, soils, and land use may occur within major watersheds. In this study, geology, soils, and land use are interrelated to such an extent that any one factor may be used in the prediction equations with similar results.

Sediment derived from upland erosion may be attributed to several sources. The most important sources of suspended sediment are the upland areas with heavier textured surface soils or the surface geologic types from which heavier textured soils develop. Specifically, within the Tobacco River watershed, 47 percent of the average suspended sediment load comes from till plain and lake plains that occupy 32 percent of the area, whereas 53 percent comes from moraine and outwash plain that occupy the remaining 68 percent of the area. A similar division was obtained, using soil association mapping units. Thus, 52 percent of the suspended sediment load comes from *SA*–7, *SA*–48, and *SA*–35 that occupy 31 percent of the area, while 48 percent comes from *SA*–16, *SA*–22, and *SA*–24 occupying the remaining 69 percent of the area.

Sediment yield also varies with land use. In the Tobacco River watershed, pasture lands are the most important sediment producers, probably because of the proximity of a large share of the pasture land to the streams. Streambottom pasture lands characteristically have many short steep slopes and comparatively short lengths of overland flow to the stream. Pasture land, occupying 18 percent of the land area, yields 42 percent of the sediment, whereas 13 percent comes from cultivated land (10 percent of area) and 45 percent comes from forest and other wild land occupying the remaining 72 percent of the area.

Of all sediment sources, eroding banks, which yield about 28 percent of the suspended sediment load, represent a source that the land manager can effectively control. In trout streams with forested drainage areas, eroding banks may be the most important source of sediment. Bank stabilization practices may, therefore, be one means of reducing sediment concentrations in these streams and improving the stream as a trout habitat.

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MEASUREMENTS OF THE SHEAR RESISTANCE OF COHESIVE SEDIMENTS

[Paper No. 23]

By FRANK D. MASCH, *associate professor, Civil Engineering, University of Texas;* WILLIAM H. ESPEY, JR., *hydraulic engineer, U.S. Geological Survey;* and WALTER L. MOORE, *chairman and professor, Civil Engineering, University of Texas, Austin*

Abstract

A rotating cylinder test apparatus designed to measure low shear stresses is described. This apparatus gives a value of the true shear stress on a sediment sample and a measurement for this shear which is independent of such uncertainties as roughness changes and boundary layer growth. The apparatus is also analyzed in view of the work on rotating cylinders by Taylor and others. A standard test procedure is outlined, and results of tests on several laboratory samples are presented.

Introduction

When concerned with stable channel design or localized scour in beds of cohesive sediments, fluvial hydraulicians often resort to selecting either a critical tractive force or a safe permissible velocity so that no undesirable erosion will occur. These methods involve choosing the hydraulic variables so that certain critical shear forces or critical velocities are not exceeded. Etcheverry (*1*), Fortier and Scobey (*2*), Lane (*3*), and others have published tables and graphs summarizing these critical shears and velocities.

While it is generally understood that the scour resistance of a cohesive sediment will depend in some fashion on the properties of the sediment, the relative significance of such parameters as density, moisture content, percentage of clay, Atterberg limits, vane shear, etc., is not fully known. In 1962, Moore and Masch (*4*) summarized much of the existing work on scour in cohesive materials and presented some test results of their own. More recently, Carlson and Enger [1] reported on tests performed at the U.S. Bureau of Reclamation in which soil samples were set into a test well flush with the bottom of a circular tank and water circulated over the sample with a rotating impeller. Critical tractive forces were measured and correlations made with several soil properties.

It is not entirely clear as to whether tractive force measurements made on soil samples set into a section of a smooth-bottom flume or tank are actually representative of the shear that would cause erosion in a cohesive bed. As the flow over the bottom suddenly encounters the soil sample, there is an abrupt change in the bed roughness and velocity distribution. This is particularly true when the surface of the sample is eroding. Under these conditions, the average tractive force is not uniformly distributed over the sample and determinations of critical shear from point velocity measurements are not necessarily representative of the shear on the sample. Tests of this type seem to be preferred, probably because they appear similar to flow in natural streams. However, the similarity may end with the flow being parallel to the bed sample and the critical shear determined from this type test no more representative for a given type bed material than that determined from jet or other dissimilar type tests.

If the relations between the critical shear stress and the various properties of a cohesive material are to be investigated, the need for a way to measure a mean shear that is constant over the entire surface of the sample is evident. The purpose of this paper is to describe a test for determining the critical shear stress at which a stiff cohesive sediment will scour. The apparatus has been designed so that the distribution of the average shear over the sample is uniform. In addition, this average shear is measured directly rather than computed from other experimental data. It should be emphasized that it is not the purpose of this paper to present design criteria for stable channels in cohesive beds.

Development of Apparatus

In 1962, Moore and Masch (*4*) briefly described an apparatus then under fabrication at the University of Texas that could be used to measure directly the shear stress on a sample of a cohesive sediment. Since first reported, the apparatus has undergone some modifications and, in the course of its development, a procedure for measuring the critical shear on a sample has been worked out.

Test Apparatus

The test apparatus was built utilizing a rotating cylinder principle common to some types of viscosimeters. As seen in figure 1, a cylinder of stiff cohesive sediment 3 inches in diameter and 4 inches long was mounted coaxially inside a larger transparent cylinder that could be ro-

[1] CARLSON, E. J., and ENGER, P. F. TRACTIVE FORCE STUDIES OF COHESIVE SOILS FOR DESIGN OF EARTH CANALS. Amer. Soc. Civil Engin., Hydraul. Div. Conf. 11, Davis, Calif. 1962.

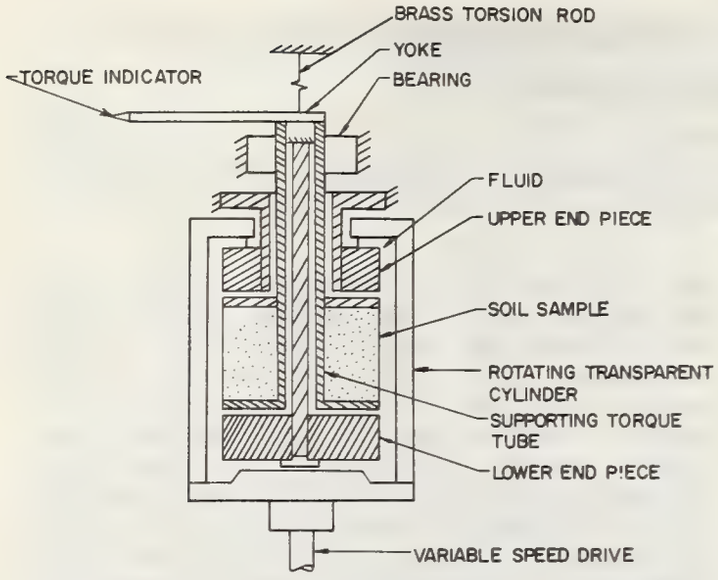

FIGURE 1. — Diagram of modified rotating cylinder test apparatus.

tated at speeds up to 2,500 r.p.m. The apparatus could be operated with either a 3.25- or a 4.20-inch diameter rotating cylinder, giving an annular spacing of either 0.125 inch or 0.60 inch, respectively. To transmit shear from the outer rotating cylinder to the surface of the soil sample, the annular space between the sample and the rotating cylinder was filled with water. Since the annular spacing between the sediment sample and the outer cylinder was constant and there were no abrupt changes in roughness on the area in shear, the stress was uniform at all points on the surface of the soil.

The sediment sample was stationary, but was mounted on a combination radial and thrust bearing so that the shear stress transmitted to its surface resulted in a slight rotation of the supporting tube. This rotation was in turn transmitted to a 0.060-inch diameter brass torsion rod 6 inches long attached through a yoke to the supporting tube. By measuring the deflection of a pointer connected to the yoke and calibrated to give the torque, the shear stress on the sample was obtained. To minimize the variation in shear stress at the ends of the cylinder, 3-inch diameter endpieces were mounted independently of the sample so that the shear applied to their surfaces would not contribute to the measured torque for the soil sample.

Nature of Flow in Apparatus

If the flow in the annular space between two concentric cylinders rotating at steady speed is considered to be parallel, i.e., only the tangential component of velocity not equal to zero, the flow can be analyzed by the Navier-Stokes equations. For the case in which the inner cylinder (radius r_1) is at rest while the outer cylinder (radius r_2) is rotating at an angular velocity of ω_2, Schlichting (*5*) gives the moment (T) transmitted by the rotating cylinder to the fluid as

$$T = 4\pi\mu h \frac{r_1^2 r_2^2}{r_2^2 - r_1^2} \omega_2 \qquad (1)$$

where h is the height of the cylinder. Equation 1 also gives the torque transmitted by the fluid to the sample. Since the shear is directly related to the torque, the shear stress on the inner cylinder or soil sample is given by

$$T_0 = 2\mu \frac{r_2}{r_2^2 - r_1^2} \omega_2 \qquad (2)$$

For equation 2 to be applicable, the flow in the annular space between the outer rotating cylinder and the fixed cylindrical soil sample must be that of a stable Couette flow. Generally, the flow in the annulus between rotating cylinders tends to be quite stable because of the effect of inertial forces. Fluid particles near the outer rotating boundary are kept from moving radially inward by large centrifugal forces, whereas those particles near the inner boundary do not move outward because of smaller centrifugal forces.

Schlichting (*5*) has compiled the work of Taylor and others on the critical Reynolds number at which the flow in an annulus between a fixed inner cylinder and a rotating outer cylinder becomes unstable. Figure 2 shows these results as a curve of the critical Reynolds number plotted as a function of the relative annular spacing s/r_2, where s is the annular spacing. For the two outer cylinders used in the soil test apparatus, the critical speeds for stable Couette flow from figure 2 are 344 r.p.m. for the smaller cylinder and 435 r.p.m. for the larger cylinder. These values are summarized in table 1.

From the data plotted in figure 2, the curve of limiting stability appears to be affected by eccentricity in the coaxially mounted cylinders. In those cases where good concentricity was maintained, the critical Reynolds number is seen to be much larger than the limiting curve passing through Taylor's data. To compare the critical Reynolds number for the soil test apparatus with the work of Taylor, the shear stress on a dummy soil sample was measured as a function of the rotational speed of the outer cylinder. The dummy cylinder, which was made of wood 3 inches in diameter and 4 inches long, was tested in the 3.25-inch diameter cylinder. This arrangement gave an annular spacing of 0.125 inch, which is almost identical to a test performed by Taylor in which the annular spacing was 0.122 inch. Results of the test on the sediment apparatus are plotted in figure 3

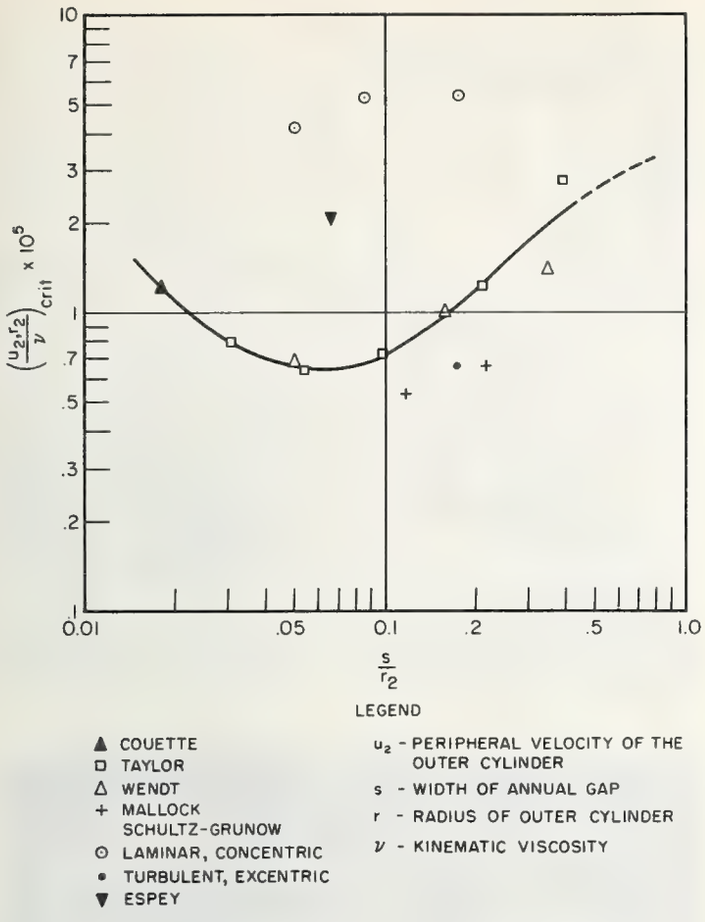

FIGURE 2. — Critical Reynolds number vs. relative annular spacing. (After Schlichting (*5*).)

where it is seen that the transition from a stable Couette-type flow to an unstable type flow occurs at a rotational speed of 680 r.p.m. This is nearly twice the value of 344 r.p.m. obtained from figure 2.

Taylor (*6*) presented his results for the 0.122-inch annular spacing in the form of two parameters, $T/\rho N^2$ and $\rho N/\mu$ where both are given in the c-g-s system of units. These results along with those for the scour apparatus are shown in figure 4.

TABLE 1.—*Critical speeds for stable flow for two cylinders used in soil test apparatus*

Item	Small cylinder	Large cylinder
Critical speed.....r.p.m...	344	435
Inner radius (r_1).....inches..	1.50	1.50
Outer radius (r_2).....inches..	1.625	2.10
Annular spacing (s).....inch....	0.125	0.60
s/r_2.....	0.077	0.285
Critical Reynolds number.....	6.6×10^4	1.4×10^4

The fact that stable conditions exist for a greater range of speeds in the scour apparatus is attributed to better concentricity. It is significant that figure 4 is analogous to the conventional skin friction-Reynolds number relation for flow over flat plates. For most cohesive sediments, the scour apparatus is operated in the unstable zone and the flow is not completely developed turbulent flow. Its operation is normally in the transition where the zone of turbulent flow probably does not extend across the full width of the annulus. This coupled with the

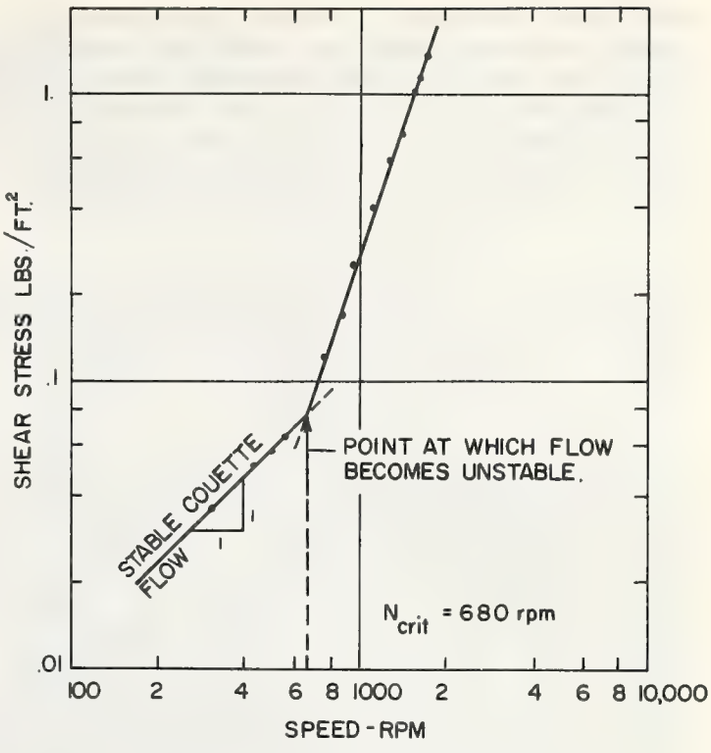

FIGURE 3. — Shear stress vs. speed.

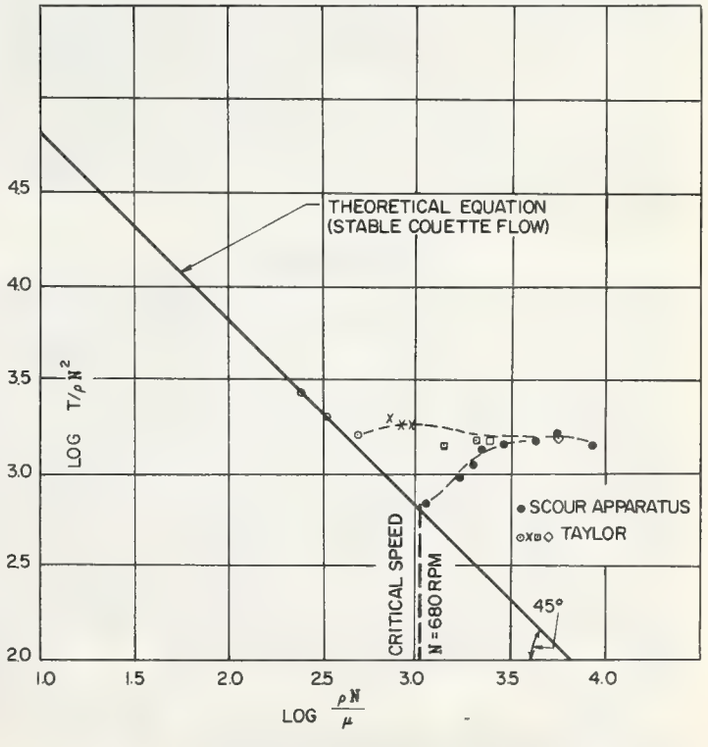

FIGURE 4. — $T/\rho N^2$ vs. $\rho N/\mu$. (After Taylor, 1960.)

fact that the inertial forces increase in the radial direction tend to make the turbulence level low and the instantaneous shear stress fluctuations on the surface of the soil relatively small. Thus the mean shear measured on the sample should be nearly equal to the maximum shear occurring at any point or at any time.

Test Procedure [2]

During the course of the development of the apparatus, two different test procedures were used to evaluate the critical shear on the surface of a sample of material. The first method consisted of loading the sample to a preselected value of shear over a period of 1 minute and allowing it to scour at this stress for 1 minute. The sample was removed from the apparatus and its weight loss determined. The sample was then replaced in the apparatus, loaded to a higher preselected value of shear in 1 minute, and allowed to scour at this higher stress for 1 minute, and the weight loss again determined. This same procedure was repeated in a systematic manner, each time with a higher value of shear stress. The weight loss was plotted as a function of the shear stress, and the range of stress at which an appreciable quantity of sediment was removed was noted.

The second method used to determine the shear at which scour began was visual. A sample of sediment was placed in the apparatus and the speed of the outer cylinder increased at a steady rate. By viewing the sediment sample through the transparent outer cylinder, it was easy to note the beginning of scour. The beginning of scour was also characterized by a sudden movement of the pointer attached to the torsion rod. When scour was observed, the deflection of the torsion rod was noted and the shear stress measured. This visual test was repeated several times and seemed to be independent of the rate of loading.

Discussion

Measurements of the critical shear causing scour on remolded samples of cohesive sediments were obtained by both test procedures outlined above. In all cases the samples were made from 100 percent Taylor marl and had moisture contents between 28 and 31 percent. The samples that were 3 inches in diameter and 4 inches long were extruded under vacuum with a Vac-Aire extruder, thus insuring that the sample was uniform and free of air voids.

Typical results of the first test procedure are shown in figure 5. At the very low shear stresses the rate of scour is very low. This

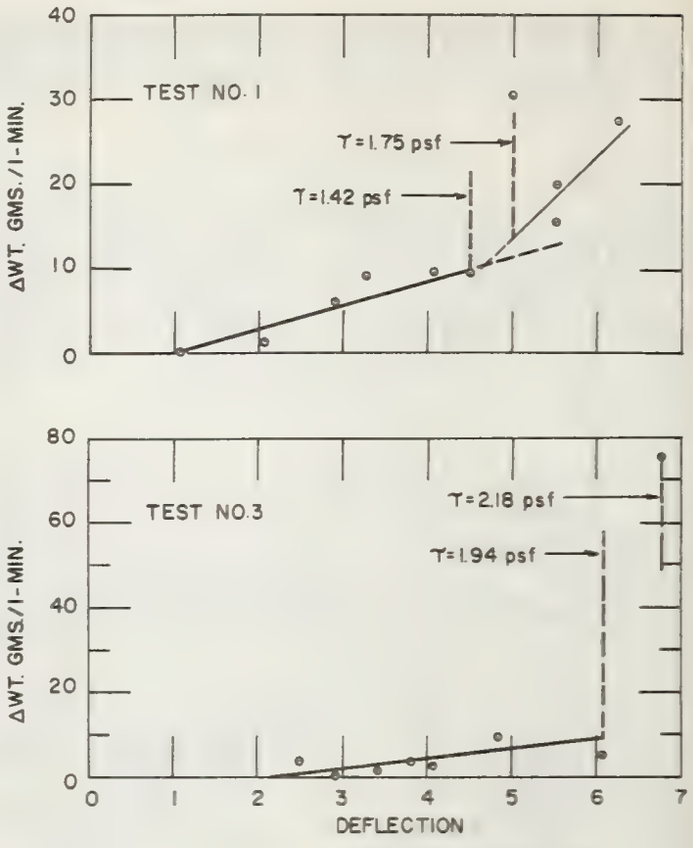

FIGURE 5. — Weight loss per 1 minute vs. deflection.

small amount of scour is attributed to a slight flaking of material from the surface of the sample and probably depends on the condition of the surface of the sample. After a certain critical shear is reached, the rate of scour is suddenly increased. This is the critical shear at which scour is considered to begin. While this type of test does not give the exact value of stress at which the scour starts, it does define limits within which this critical shear must fall. The results of a series of these type tests are tabulated in table 2.

TABLE 2.—*Critical shear stress by visual test and by weight loss per minute*

Test No.	Critical shear stress range from first test method	Test No.	Critical shear stress from visual test	Moisture content
	Pounds per square foot		*Pounds per square foot*	*Percent*
1. . . .	1.42–1.75	1. . .	1.74	28
2. . . .	1.94–2.18	2. . .	1.96	31
3. . . .	1.02–1.58	3. . .	1.48	31
4. . . .	1.35–1.45	4. . .	1.74	31
5. . . .	1.47–1.52	5. . .	1.58	31
		6. . .	1.82	28
		7. . .	1.74	30

Several tests that use the visual method for determining critical scour were run on the same material. These results are also summarized in table 2. Again, at the lower speeds it was pos-

[2] ESPEY, W. H., JR. A NEW TEST TO MEASURE THE SCOUR OF COHESIVE SEDIMENT. Master's thesis. On file, University of Texas library, Austin.

sible to observe what might be called a washing of the surface, i.e., the flaking of small soil particles from the surface of the sample. As the speed and the shear stress were increased, the sample was observed through the transparent cylinder until the critical shear was reached, at which time appreciable quantities of sediment came loose from the sample and the water in the annulus became cloudy. After a large quantity of material had scoured from the sample, it no longer conformed to a sample with a uniform shear stress distribution. At the point where scour occurs, separation of the flow develops, producing intense turbulence in the scour hole and downstream from it. The form drag produced by the hole results in tangential forces and thus increases the shear in the hole resulting in a reduction in the shear on the rest of the sample.

If the nature of the material tested is considered, the results obtained with the different test procedures are in fair agreement. In most cases, the visual test results fall within the range of shears obtained with the first test method. There is not enough variation in moisture content to tell whether any relation exists between it and the shear stress.

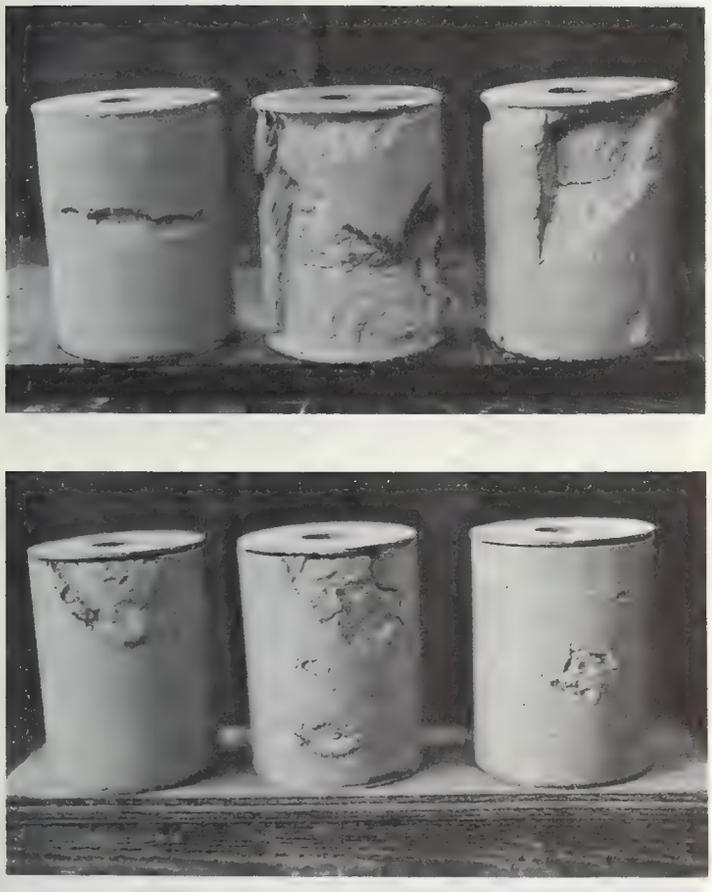

FIGURE 6. — Typical scour patterns.

Systematic failure of the various soils tested was not apparent. Figure 6 shows several typical scour patterns resulting from both types of scour tests. Those samples showing large amounts of scour were tested according to the first method outlined and were subjected to scour at shears greater than the critical value. Those samples showing only small amounts of scour were tested by the visual method. The failure of the sample at several different points supports the contention that the shear stress was relatively uniform over the entire sample.

Conclusions

Based on the results of the investigation outlined above, several conclusions appear to be justified. The rotating cylinder apparatus has been developed and test procedures outlined that enable the critical shear stress at which a given stiff cohesive sediment scours to be obtained. The test procedure is relatively simple and if the nature of the sediment tested is considered, it appears to give consistent results. The level of turbulence is believed to be low so that the average shear measured on the sample when erosion starts should be very near to the actual stress causing erosion, based upon studies of flow in the annular spacing between the soil sample and the rotating outer cylinder.

It now appears feasible to investigate the effect of the various properties of the sediment on the critical shear causing scour. Until a better understanding of the nature of scour is obtained, it is believed that basic studies of this type can best progress by performing tests on remolded sediments so that some control can be maintained over its properties.

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SURFACE RUNOFF AND EROSION AS AFFECTED BY SOIL RIPPING

[Paper No. 24]

By E. J. DORTIGNAC, *forester (research)*, and W. C. HICKEY, JR., *range conservationist (research)*, *Rocky Mountain Forest and Range Experiment Station, Forest Service, Albuquerque, N. Mex.*[1]

Sedimentation is the most critical watershed problem in the arid and semiarid Southwest. It is particularly severe in the Rio Grande basin in New Mexico, where sedimentation depletes reservoir capacities, aggrades river channels, increases the need for maintenance of irrigation systems, damages land and crops, wastes water by evaporation and by evapotranspiration of nonbeneficial vegetation, and contributes to downstream salinity. The worst offender is the Rio Puerco tributary — since 1885 an estimated 600,000 to 800,000 acre-feet of soil has washed into the Rio Grande. Even now, the Rio Puerco, which yields only about 5 percent of the measured runoff above Elephant Butte Reservoir, produces almost half of the measured sediment inflow to the main channel. This damaging sedimentation must be controlled or reduced.

Past misuse and mismanagement has resulted in a deteriorated vegetation cover on the lower lying foothills, mesas, escarpments or bluffs, and valley floors. Over large areas, the once important and highly productive dominant grasses, such as alkali sacaton (*Sporobolus airoides* Torr.), galleta (*Hilaria jamesii* Torr.), black grama (*Bouteloua eriopoda* Torr.), Indian ricegrass (*Oryzopsis hymenoides* (Roen & Schult.) Ricker), and blue grama (*Bouteloua gracilis* (H.B.K.) Lag.), have been largely replaced by less desirable grasses and shrubs. The major change in the natural vegetation of these rangelands has occurred since 1860. Yet, deterioration of these lands had already been observed by explorers of the mid-nineteenth century. Abert (*1*), Simpson (*6*), and Whipple (*9*) observed gullies in the main Rio Puerco channel between 1846 and 1853.

The deterioration of the vegetation has been accompanied by a great reduction in ground cover, leaving most of the land with exposed or bare soil (fig. 1). Surveys and field studies indicate that bare soil and rock comprise 90 to 100 percent of the land area on deteriorated sites.

The resultant harsh microclimate (particularly on south and west exposures) makes rehabilitation of these lands a most difficult task. Management and proper grazing use cannot be expected to restore these lands to full productivity without supplemental mechanical treatments. Moreover, a good vegetation cover for

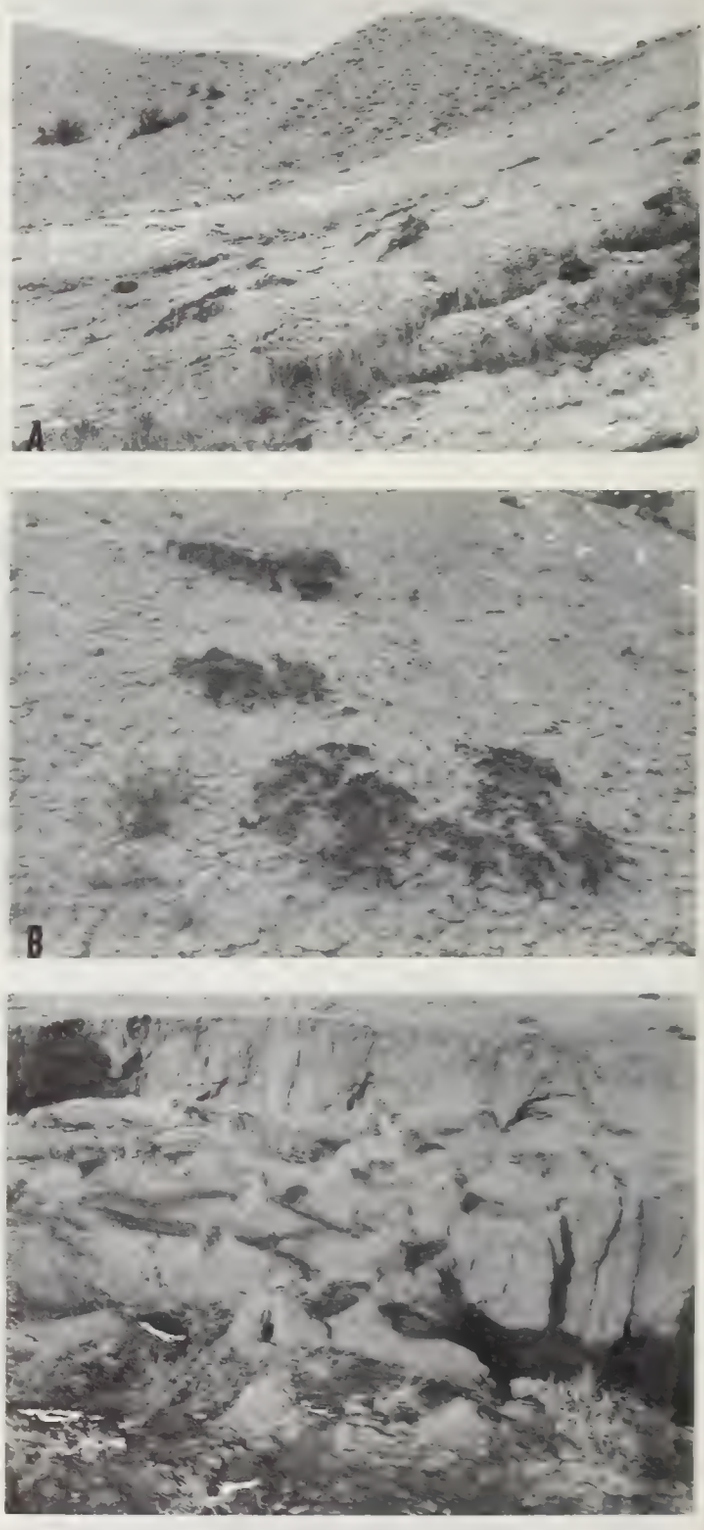

FIGURE 1. — Loss of vegetation and deterioration of the range: *A*, Dead alkali sacaton on shale hill; *B*, root crowns of dead alkali sacaton; and *C*, gully on alluvium downslope from the shale hill.

[1] A cooperative study between the Bureau of Land Management, U.S. Department of the Interior, and the Forest Service, U.S. Department of Agriculture.

holding soil in place cannot be attained until desirable species become established in sufficient numbers to provide a continuous source of seed dissemination over these lands. Experience has shown that re-establishment of vegetation on deteriorated arid and semiarid rangelands is a slow process and that many years are required for full recovery.

Reseeding grass and browse has usually failed on the Upper Rio Grande basin rangelands below the big sagebrush (*Artemesia tridentata* Nutt.) belt or where average annual precipitation is less than 12 or 13 inches. Improvement of the severe microclimate on deteriorated sites in this semiarid climate is an essential first step in any program of vegetation restoration. An increase in soil moisture and a reduction in air and soil temperatures are the most important elements to be considered.

Improvement of the soil moisture regime may be accomplished by increasing the water retention capacity, the infiltration capacity, or both. Several soil treatments, such as basin listing, contour trenching, and pitting, are recognized methods of increasing the retention capacity and prolonging the time for infiltration. Depending on soil type and condition, this type of land treatment may or may not increase the rate of water intake. Yet the initial retention of water increases the time for infiltration, at least until the basins or soil openings become filled with sediment.

An increase in soil moisture usually results in greater production of forage, crop, or trees. Several reports indicate range pitting with modified disks that gouge out small, closely spaced basins has improved forage production and grazing capacity (*2, 3, 5*). Also, Rauzi (*4*) reported higher water infiltration on pitted rangeland. These studies indicated that 50 to 100 percent more water infiltrated in pitted areas than in adjacent nontreated areas and that a newly formed pit holds 0.3 inch of rain. Measurements of the average size of pits made by several types of mechanical equipment in New Mexico indicate that basins when first formed hold 0.25 to 0.50 inch of water. Subsequent sediment-laden surface runoff may soon fill the pits with soil and reduce their effectiveness for water storage. However, the establishment of more desirable hydrologic plant species or an increase in the ground cover as a result of pitting usually contributes to higher infiltration capacities. Moreover, on certain soils with impermeable layers near the surface, a rotary pitter with a long tooth can loosen and open this layer to water intake and prolong the effectiveness of the pit.

Valentine (*8*) reported that five selected types of structures to check runoff water from semidesert range (New Mexico Experiment Station's Experimental Ranch) failed to improve the vegetation cover on the sites where installed. One of the treatments included large ripper furrows made with a road ripper. This implement dug a broad, flat furrow about 6 inches deep and 24 to 30 inches across. Soil factors, such as erodibility and low moisture retention capacity, were thought to be responsible for the lack of vegetation improvement under all treatments tested.

Soil ripping increases the water storage capacity of the land until the fissures become filled with washed-in or blown-in soil and until weathering levels the roughened surface (fig. 2). Soil ripping may derive its greatest hydrologic benefit by increasing the water absorbing

FIGURE 2. — Temporary water-storing dams or terraces formed in the soil-ripping operation.

area and by allowing surface runoff water to penetrate directly into the less permeable subsoil. This latter condition is extremely important when shallow soils are underlain with

impervious parent material or developed B horizons.

The reported study is an attempt to evaluate the effectiveness of soil ripping with a Jayhawk Soil Saver on surface runoff, erosion, and vegetation response.

Location of Study

The study area is a seriously deteriorated site in the Rio Salado drainage that enters the Rio Jemez, 2 miles south of San Ysidro, N. Mex. This site probably represents the poorest range and hydrologic condition in the Rio Jemez and adjacent Rio Puerco basins (fig. 3). Ground cover in the summer of 1958, when the site was selected, averaged less than 2 percent. Annual forbs comprise the main vegetation. There were few living alkali sacaton plants, but numerous dead plants indicated this grass species previously covered most of the shale hills, colluvium, and alluvium (fig. 1).

Soils are derived from Mancos shale, an Upper Cretaceous marine deposit of high salinity. This parent material is covered by 10 inches to 8 feet of soil on the northeast exposure and mostly exposed shale on the southwest aspect. Mancos shale disintegrates quite rapidly when immersed in water. Laboratory tests showed that 1- to 2-pound samples of hard shale, obtained with a pick, dispersed completely within 10 minutes after immersion in a vessel containing tap or distilled water. This instability is conducive to rapid weathering when this shale rock is exposed to the elements of climate.

The clay mineral has been identified as illite, a hydrous mica. Derived soils are silty clay and silty clay loams. The alluvium on both aspects is cut by numerous fingering gullies, a result of the past high surface runoff over the area (fig. 4). Vertical gully walls are common in this soil type.

Annual precipitation averaged about 8.3 inches during the 3 years. About 70 percent of the precipitation was received mostly in high-intensity rainstorms (maximum rate recorded was 4.5 inches per hour for 3 minutes) between April 1 and December 1, while the balance was largely snowfall (table 1).

FIGURE 3. — General view of southwest aspect and close-up of shale soil of study area.

FIGURE 4. — General views of northeast aspect, the ripping site, and fingering gullies.

TABLE 1.—*Precipitation at experimental site in the Rio Salado drainage, 1958-61*

Year	December 1 through March 31	April 1 through November 30
	Inches	*Inches*
1958–59	1.66	5.31
1959–60	3.89	6.45
1960–61	1.87	5.74
Average	2.47	5.83

The long-term average annual precipitation, estimated from the isohyetal map of the Rio Puerco, is about 10 inches (*7*). Accordingly, below average precipitation occurred during this study.

Method of Study

Soil Ripper (Jayhawk Soil Saver)

The soil ripper (Jayhawk Soil Saver) (fig. 5)[2] was developed and first used in Kansas. A revolving auger behind the chisel rotates clockwise as the ripper is pulled through the soil profile. Field excavations have shown that this ripper forms rather large openings in the subsoil which provide an appreciable increase in the water-holding capacity and in the water-absorbing area.

The Jayhawk Soil Saver required a D-8 tractor on these and similar soils. The depth of ripping varied from 24 to 30 inches, depending on soil bulk density, soil moisture, and depth to the parent material. Soil rips were spaced 7 feet apart.

The soil ripper in action (fig. 5) and the land surface after treatment (fig. 2) indicate the soil treatment possibilities with this equipment. Some residual vegetation is destroyed during soil ripping, but this loss may be more than offset by the grass recovery (fig. 6).

FIGURE 5. — Soil ripper (Jayhawk Soil Saver) pulled by a D–8 tractor.

[2] More recently Bureau of Land Management technicians have modified the Jayhawk Soil Saver by adding a range seeder and two duckfoot cultivators on both sides of this soil ripper. The present study covers ripping with the unmodified Jayhawk Soil Saver.

FIGURE 6. — Grass recovery 1 year after treatment with the soil ripper; untreated area at extreme left.

In field trials conducted in Wyoming, Rauzi and Lang (*5*) found that pitting with the offset disk removed approximately one-third of the existing vegetation, but the increased volume of growth on the remaining plants exceeded the production from the nontreated areas. Less vegetation is destroyed in soil ripping than in soil pitting with offset disks.

Field Installations

Surface runoff plots were installed on the upper and lower slopes of a northeast and a southwest exposure below a shale hill (fig. 7). Plots on the upper slope were placed near the base of the shale hills, whereas those on the lower slope were about 200 to 300 feet above the gullies (fig. 8). Depth of soil increases in proceeding downslope from the shale hill toward the gullies. Slope gradient averages 10 percent

FIGURE 7. — Runoff plots on southwest aspect below the shale hills.

FIGURE 8. — Gully below the shale hills and runoff plots.

on the northeast aspect and 6 percent on the southwest exposure.

The design provides for a comparison of the effect of soil ripping by exposure and slope position. In addition, two closures to exclude rabbits and livestock (proof exclosures) were constructed at each of the above positions to provide a replication of these treatments by slope position and exposure. Four separate treatments, representing untreated soil (the control), soil ripping, soil ripping with seeding to alkali sacaton, and soil ripping with seeding to chamiza, or fourwing saltbush (*Atriplex canescens* (Pursh) Nutt.) were installed for each above-mentioned condition (fig. 9).

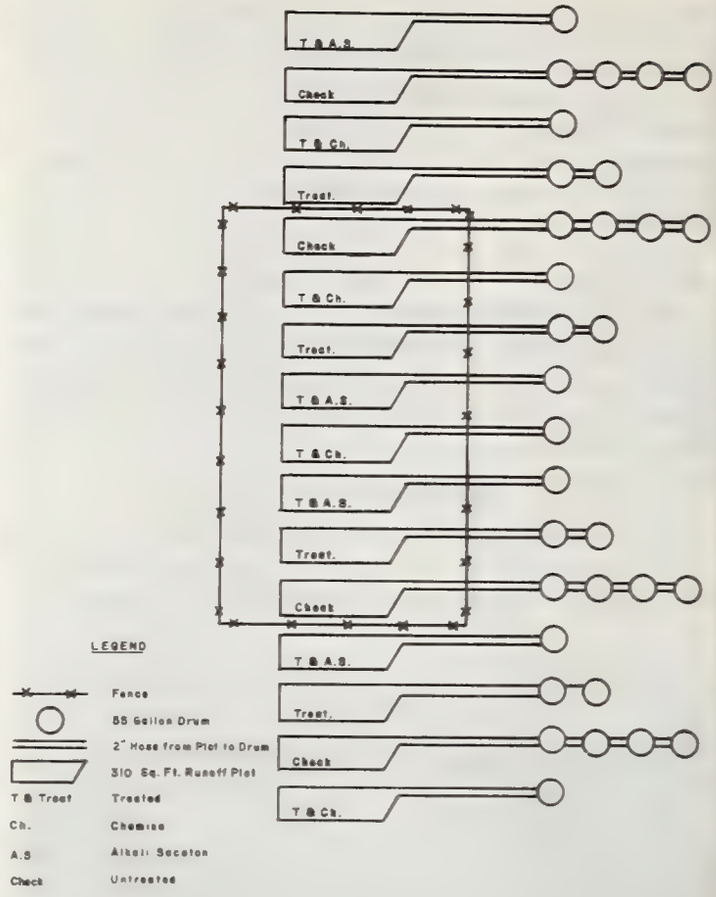

FIGURE 9. — Plot layout for one slope and one exposure.

In all, 64 surface runoff plots were installed at this location. Plots are 10 x 30.5 feet, with the length parallel to the slope. Plot materials and construction details have been described.[3]

The overall plot layout was staked in the field prior to soil ripping, which was done on the contour. The soil ripper was lifted out of the soil before crossing the check or "no treatment" plot zones. Timbers were laid to prevent soil compaction by the tractor and the ripper on the untreated area. A 10-foot buffer or isolation strip separates the individual plots.

It was necessary to seal the soil fissures formed by the Jayhawk Soil Saver on both sides of each plot.[4] This was accomplished by using a steelplate ($\frac{3}{8}$ x 8 x 40 inches) with a piece of $\frac{3}{8}$-inch angle iron welded to the top (fig. 10). The pointed end of this plate was placed against the outside border of the plot and pounded into the ripped zone with an 8-pound hammer. Loose soil was shoveled from the soil rip adjacent to the steel plug and tightly tamped back in place with the head and handle of the sledge hammer. A handyman jack was used to extract the steelplate without disturbing the plot borders nor the tamped soil.

[3] GARCIA, GEORGE, HICKEY, W. C., JR., and DORTIGNAC, E. J. AN INEXPENSIVE RUNOFF PLOT. U.S. Forest Serv. Res. Note RM–12, 8 pp., illus.

[4] This method was first tested by sealing various 3-foot lengths of ripped soil. After the sealing was completed, 110 gallons of water were poured into each length of rip. Forty-eight hours later the rips were dug out at each end of the 3-foot sealed portions. No leakage was noted in any of these preliminary tests nor in subsequent examinations of soil profiles outside runoff plots after rainstorms.

FIGURE 10. — Sealing the soil rips at runoff plot borders.

Four rain gages, two recording and two standard, were installed at the start of this study. Two more gages were added later. Six gages were needed, because precipitation varied appreciably for certain storms (fig. 11). Despite the variation in precipitation measured between gages for individual storms, the seasonal catch did not vary greatly between gages on the same aspect.

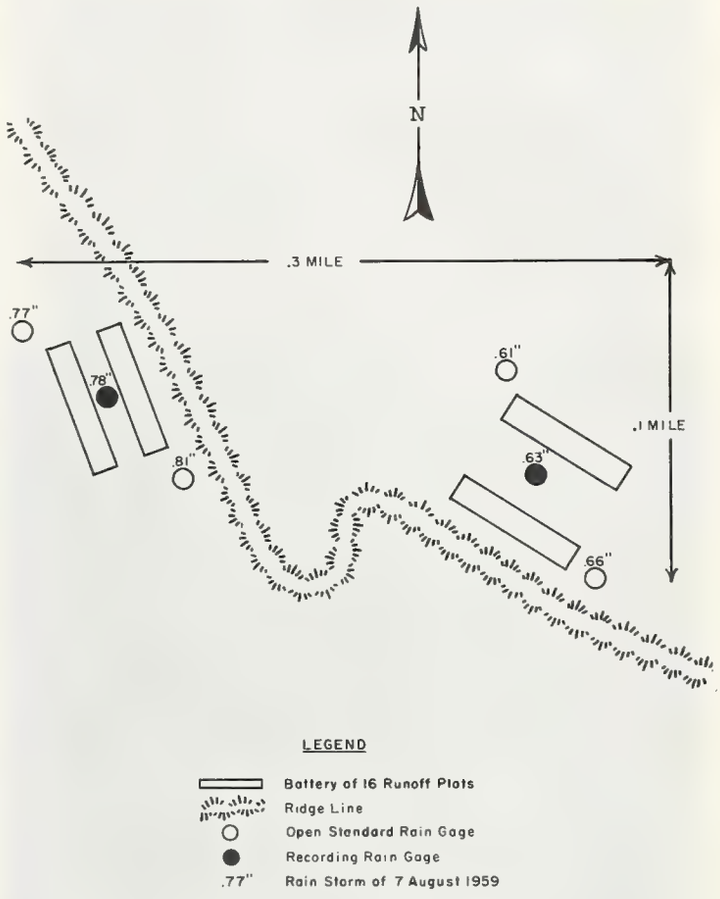

FIGURE 11. — Study layout and location of rain gages. Note the variation in rainfall measured after a single summer rainstorm.

Results

Vegetation

Attempts to establish alkali sacaton and chamiza were mostly unsuccessful. These species were seeded in the spring, summer, and fall of 1959, 1960, and 1961. Seeds germinated when favorable moisture conditions prevailed but died in the subsequent excessive heat, drought, or severe overwinter conditions. On several occasions, the excessive surface runoff during torrential rainstorms washed the surface soil and germinated seedlings into the collection troughs and barrels.

Straw was used for mulching on two soil-ripped plots seeded in the summer of 1961. Al-

kali sacaton became established on one plot and chamiza on the other. These two plots were covered with snowfence to provide half-shade in late August and September.

Seedlings survived the winter with no protection except the small amount of residual straw that provided a partial cover. By May 1962, the dry soil and strong winds were threatening the survival of these established plants. Soil deposition from wind had already covered some of the new plants.[5] Additional straw mulch was placed over these two plots and on the bare soil to the windward. In August 1962, an excellent stand of alkali sacaton had become established on one plot and chamiza on the other (fig. 12). The study has not progressed long enough to provide an indication of the effect of this newly established vegetation on surface runoff and erosion.

Native vegetation improved slightly over the entire site during the 3 years. No significant differences in vegetation were measured between the various conditions sampled at the start and end of the study, except on the two plots successfully seeded.

FIGURE 12. — One-year-old alkali sacaton plants appear permanently established.

[5] Soil movement by wind action was most serious during the 1961-62 fall-winter-spring period. By May 1962, windblown soil had filled the 0.3-cubic-foot capacity of runoff collection troughs, necessitating removal of soil before the start of the summer rains.

Surface Runoff

First year after treatment

Overwinter (1958-59) and early-spring precipitation did not produce measurable runoff from the check or untreated plots on the upper slope of either aspect. These were the first plots to be completely installed. The first rainstorm that produced measurable surface runoff occurred on May 26, 1959 (table 2).

TABLE 2.—*Rainfall and surface runoff from upper slope untreated plots, May 26, 1959*

Aspect	Rain	Surface runoff	Portion of rain as runoff
	Inch	*Inch*	*Percent*
Northeast	0.58	0.248	43
Southwest	.26	.150	58

A complete reduction (100 percent) in surface runoff was attributed to soil ripping because no runoff was measured on any of the treated plots. This was an important reduction, as about one-half of the rainfall produced surface runoff.

All plots were operating before the next rain, which fell on June 21. This storm varied between 0.26 inch and 0.30 inch among the four rain gages then installed. About 32 percent of this rain ran off the untreated plots on the northeast exposure, and 8 percent ran off on the southwest aspect. No runoff was yielded from any of the soil-ripped plots.

Nine additional rainstorms produced surface runoff in 1959, the last storm occurring on October 30 and 31. Five of these storms produced appreciable quantities of runoff on untreated plots (table 3).

TABLE 3.—*Total rainfall and total surface runoff from untreated plots for period June 21 through October 31, 1959*

Aspect and slope	Rainfall	Surface runoff	Portion of rain as runoff
	Inches	*Inches*	*Percent*
Northeast; upper	4.35	1.516	35
Northeast; lower	4.35	1.972	45
Southwest; upper	4.45	.311	7
Southwest; lower	4.45	.464	10

Surface runoff varied from 9 to 72 percent of the total measured rainfall, depending on the size of storm and rainfall intensity. To illustrate, the three largest rainstorms (0.94, 0.79, and 0.70 inch) on the northeast aspect produced 59 and 74 percent of the total annual runoff from check plots on the upper and lower slopes, respectively. But measured runoff was very similar to the quantity of rainfall received at rates

exceeding 0.25 inch per hour for all but the last storm. Rainfall intensities were much lower during the prolonged October 30-31 storm — only 0.02 inch of rain fell at rates exceeding 0.25 inch per hour. Surface runoff of 0.47 and 0.56 inch for the upper and lower slopes, respectively, approximated the total rainfall exceeding 0.10 inch per hour. Surface runoff amounted to 51 percent of storm rainfall on the northeast exposure.

Surface runoff was much less from plots installed on the southwest aspect, which contains mostly shale parent material. Two explanations are given for this apparently higher infiltration rate on the much poorer soil and more xerophytic site. The shale material develops large cracks in drying due to shrinkage (fig. 13). This network of fissures reacts similarly to the mechanically soil-ripped areas in regard to water intake. Also, soil ripping on the adjacent surrounding area may have facilitated soil cracking on the untreated plots and caused soil piping, a type of subterranean or tunnel erosion (fig. 14). Soil piping was first observed on the southwest aspect.

Regardless of this unanticipated finding, soil ripping caused an appreciable reduction in surface runoff on both aspects during the first year following treatment (table 4).

TABLE 4.—*Surface runoff on soil-ripped and untreated plots, 1959–60 season*

Aspect and slope	Surface runoff		Reduction in surface runoff
	Untreated	Ripped	
	Inches	*Inches*	*Percent*
Northeast; upper	1.764	0.079	96
Northeast; lower[1]	1.944	.029	99
Southwest; upper	.532	.004	99
Southwest; lower[1]	.355	.027	92

[1]Excludes the May 26, 1959, rainstorm—plots not installed.

Second year after treatment

During the second summer after soil treatment there were 13 storms, each with less than 0.2 inch of rain. Surface runoff was not produced. However, the 5-day autumn rainstorm (October 15-19) produced surface runoff. Soils were extremely dry at the start of the rainstorm, and surface runoff from untreated plots was much less at the beginning than during the latter part of the storm when soils were wet (table 5).

TABLE 5.—*Surface runoff on untreated plots during rainstorm October 15 to 19, 1960*

Aspect	October 15-17			October 19		
	Rain	Surface runoff	Portion as runoff	Rain	Surface runoff	Portion as runoff
	Inches	*Inch*	*Percent*	*Inch*	*Inch*	*Percent*
Northeast	2.48	0.124	5	0.90	0.352	39
Southwest	2.54	.056	2	.88	.264	30

Reduction in surface runoff caused by ripping on the southwest aspect was much greater during the latter part of the storm (table 6).

Apparently, after 2.5 inches of rain in 3 days the soil cracks on the untreated plots of the southwest aspect were filled by soil swelling and by deposition of eroded soil.

The portion of rain as surface runoff on the northeast exposure was much less in 1960 than

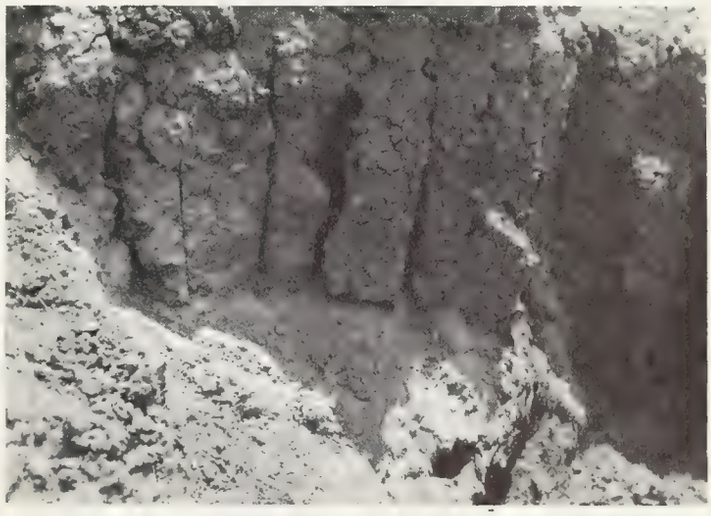

FIGURE 13. — Characteristic vertical cracking of shale material into hexagonal columns.

TABLE 6.—*Reduction in surface runoff on soil-ripped plots during rainstorm October 15 to 19, 1960*

Aspect and slope	Reduction in surface runoff	
	October 15-17, 1960	October 19, 1960
	Percent	*Percent*
Northeast; upper	73	77
Northeast; lower	99	97
Southwest; upper	26	67
Southwest; lower	44	69

in 1959, but it was quite similar on the southwest aspect (table 7).

Third year after treatment

In 1961, eight rainstorms produced surface runoff; the first occurred on July 3 and the last on October 30. This runoff was markedly reduced by soil-ripping even in the third year after treatment (table 8). Total surface runoff and the reduction caused by soil ripping is given by aspect and slope position.

Reduction in runoff varied from 61 to 100 percent for individual storms. A somewhat greater percent of reduction in runoff was measured from the smaller storms (table 8).

TABLE 7.—*Total rainfall and surface runoff on untreated plots, 1960*

Aspect and slope	Rainfall	Surface runoff	Portion of rain as runoff
	Inches	*Inch*	*Percent*
Northeast; upper	3.38	0.219	6
Northeast; lower	3.38	.733	22
Southwest; upper	3.42	.386	11
Southwest; lower	3.42	.254	7

TABLE 8.—*Surface runoff on treated and untreated plots, July 9–October 30, 1961*

Aspect and slope	Total surface runoff		Reduction in surface runoff	Reduction in runoff for storms of—	
	Untreated	Treated		Less than 0.10 inch	0.10 to 0.52 inch
	Inches	*Inch*	*Percent*	*Percent*	*Percent*
Northeast; upper	1.404	0.339	76	78	76
Northeast; lower	1.707	.266	84	96	84
Southwest; upper	.475	.061	87	90	86
Southwest; lower	.687	.120	83	94	81

The portion of rainfall yielded as runoff from untreated plots in 1962 varied between aspects and positions (table 9).

TABLE 9.—*Total rainfall and surface runoff from untreated plots, 1962*

Aspect and slope	Rainfall	Surface runoff	Portion of rain as runoff
	Inches	*Inches*	*Percent*
Northeast; upper	4.65	1.40	30
Northeast; lower	4.65	1.71	37
Southwest; upper	4.41	.48	11
Southwest; lower	4.41	.69	16

Soil Erosion

In 1959 and 1961, soil losses from untreated plots on the northeast aspect were much greater than from the poorer site on the southwest exposure. In contrast, the late fall rainstorm of October 15-19, 1960, resulted in similar amounts of erosion from both aspects. Table 10 compares the annual measured soil losses from treated and untreated plots. The reduction in erosion averaged 86 percent in 1959, 73 percent in 1960, and 30 percent in 1961. However, erosion from land slopes was relatively small from all conditions sampled. Even the maximum annual soil loss of 4,152 pounds per acre from the upper slope on the northeast aspect is considerably less than might be expected from these deteriorated sites. But plots 31 feet long do not accurately measure erosion losses — probably less than one-third of the actual soil losses occurring above the gully system on this site. On the other hand, surface runoff per unit-area on these plots is much more representative of runoff over the larger site area.

Summary and Conclusions

By 1963, approximately 30,000 acres of public domain in the Rio Puerco has been treated with the Jayhawk Soil Saver, commonly called a soil ripper, and seeded to grass and browse species. The maximum slope treated is about 20 percent, but slopes as great as 30 percent might be so treated.

During the 3-year study, surface runoff did

TABLE 10.—*Comparison of soil losses by topographic positions for soil-ripped and untreated plots, 1959–61*

Year, aspect, and slope	Untreated	Ripped	Reduction
1959	*Pounds/acre*	*Pounds/acre*	*Percent*
Northeast; upper	2,218	110	95
Northeast; lower	1,912	100	95
Southwest; upper	300	74	75
Southwest; lower	614	131	79
1960			
Northeast; upper	76	27	64
Northeast; lower	200	7	96
Southwest; upper	252	90	64
Southwest; lower	81	27	67
1961			
Northeast; upper	4,152	1,877	55
Northeast; lower	2,480	1,350	46
Southwest; upper	1,763	1,634	7
Southwest; lower	1,888	1,691	10

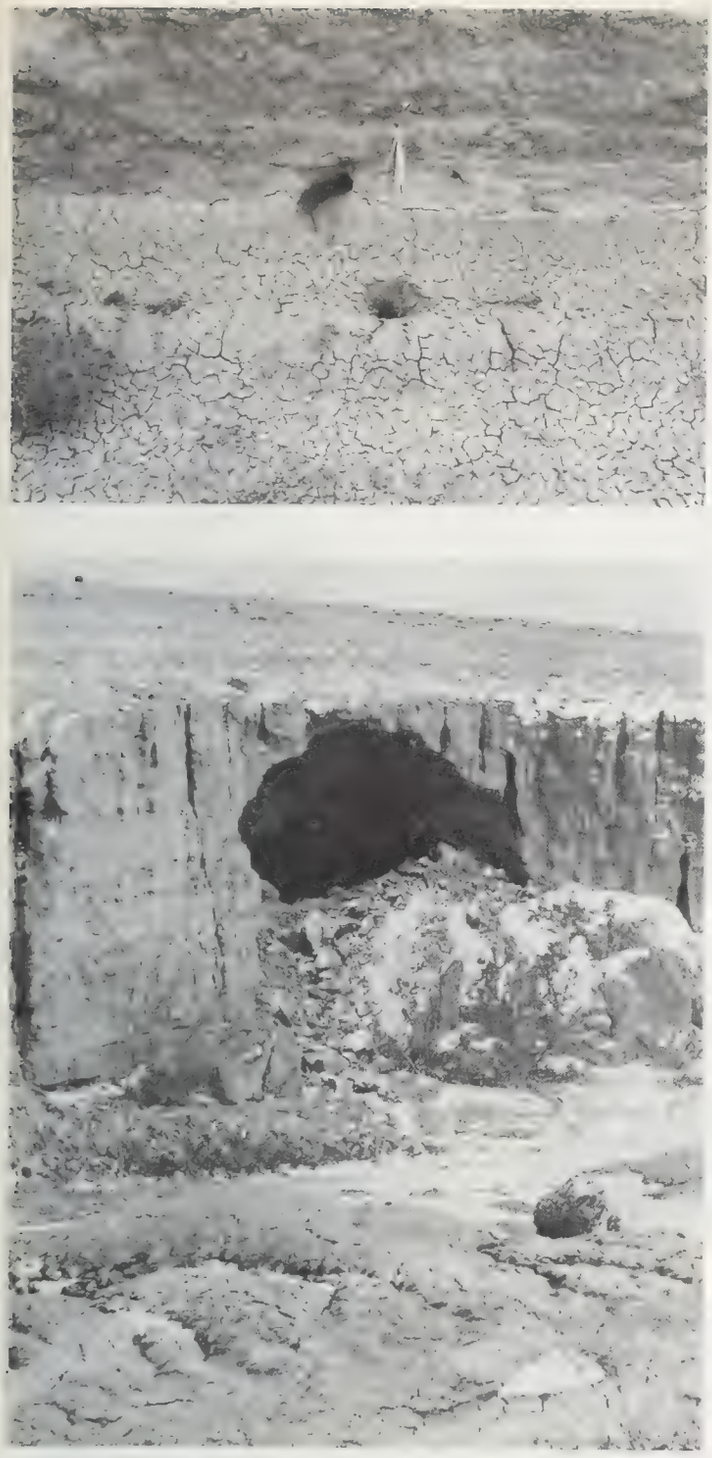

FIGURE 14. — Soil pipe openings on a side slope and in a gully wall representing the pipe inlet and outlet.

not occur before May 26 nor after October 31. During these years measured runoff from untreated or check plots averaged 22 percent of the runoff-producing rain received in the period between May 26 and October 31, 32 percent on the north aspect and 11 percent on the south exposure. But surface runoff from individual storms varied from 0 to 93 percent of the total storm rainfall, depending on quantity of rainfall, rainfall intensity, and antecedent soil moisture conditions. However, annual runoff was only 10 percent of annual precipitation.

Soil ripping with the Jayhawk Soil Saver reduced surface runoff about 97 percent the first year. The effect of this treatment persisted even without vegetation improvement during the 3 years. In the third year following the soil-ripping operation, an 83-percent reduction in surface runoff was experienced. Erosion was likewise reduced, amounting to 86 and 30 percent for the first and third years following the soil-ripping operation.

Soil ripping is effective in reducing surface runoff and erosion from deteriorated rangelands in this area.

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A SUMMARY OF THE WORK OF THE INTER-AGENCY SEDIMENTATION PROJECT

[Paper No. 25]

By FRED S. WITZIGMAN, *hydraulic engineer, U.S. Army Engineer Division, Missouri River*

Abstract

The Inter-Agency Sedimentation Project was initiated in 1939, mainly to develop improved sediment sampling instruments and techniques for measurement and analysis of sediment loads in streams. In 1956, the purpose of the project was expanded to include the solution of sedimentation problems that are of common concern to Federal agencies on the Subcommittee on Sedimentation of the Inter-Agency Committee on Water Resources, with special emphasis on methods of sampling automatically.

Manually operated sediment samplers developed and currently recommended for field use include: (1) three depth-integrating suspended-sediment samplers that collect samples continuously from the stream as they are lowered from the surface to the bed and raised back to the surface; (2) two point-integrating suspended-sediment samplers with electrically operated valves; and (3) three bed-material samplers.

Instruments have been developed, such as the single-stage sampler that is in widespread use and the pumping samplers that are in the field-testing stage, for obtaining samples or sediment information automatically from flashy streams when no observer is present.

Two sediment-size analyzers have been developed. The bottom-withdrawal tube is a sedimentation device for size analysis of sediments finer than 0.7 mm. The visual-accumulation tube is a sedimentation device for the particle-size analysis of sand samples.

Methods for determining concentration and particle size gradation which are being explored are electronic sensing, turbidity, ultrasonic sensing, and nuclear.

History and Program of the Project

Several agencies of the United States Government organized an Interdepartmental Committee in 1939 to study problems in collecting sediment data and to develop, improve, and standardize methods and equipment for determining the quantity and character of sediment carried by streams. The initial project was under the general supervision of E. W. Lane of the Iowa Institute of Hydraulic Research. In April 1946, the activities and functions of the Committee were transferred to the Subcommittee on Sedimentation of the Federal Inter-Agency River Basin Committee. One of the objectives of this Committee is the coordination of the hydrologic activities of the Federal Departments through the assistance of several subcommittees. In June 1948, the project was transferred from the Iowa Institute of Hydraulic Research to the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota in Minneapolis, Minn. In 1955, the name of the parent Committee was changed to the current designation, Inter-Agency Committee on Water Resources.

In 1956, the Subcommittee on Sedimentation adopted a Guidance Memorandum that outlined the program and organization of the present Inter-Agency Sedimentation Project. The basic purpose of the project is to solve sedimentation problems of common concern to agencies that are represented on the Committee. Since 1956, major emphasis has been on development and improvement of equipment for the automatic collection and analysis of sediment samples.

The executive direction of technical phases of the project is the responsibility of a Technical Committee, whose membership is made up of representatives from Federal agencies actively interested in sedimentation problems. The Committee provides technical advice and assistance to the project staff, who carry out the development, testing, and calibration of instruments, preparation of technical reports, and other operational phases of the project. The agencies actively cooperating in the project and currently represented on the Technical Committee are: Army Corps of Engineers, Geological Survey, Bureau of Reclamation, Agricultural Research Service, Public Health Service, Forest Service, Tennessee Valley Authority, and Soil Conservation Service. Results of the cooperative study to date are incorporated in a series of technical reports listed in the appendix. Many sediment sampling devices and analytical methods have been developed by the project.

The following personnel are currently active on the project: Frederick S. Witzigman, who prepared this paper, Russell P. Christensen, and Martin E. Nelson of the U.S. Army Corps of Engineers; and Byrnon C. Colby, project engineer, who reviewed this paper, Thomas F. Beckers, and John V. Skinner of the U.S. Geological Survey.

This paper was prepared to summarize major accomplishments of the project. The work of the project from 1939 to the present time may be divided into four phases. The three that relate to the development of sampling or analyzing devices are discussed in this paper under "man-

ual sediment samplers," "automatic samplers," and "sediment analyzing devices." Work under a fourth title "theories and procedures" is not directly and closely related to the development of specific methods or instruments and is not discussed in this paper but is covered in the reports listed on page 176.

Manual Sediment Samplers

A series of suspended-sediment and bed-material samplers (some are shown in fig. 1) that are operated manually have been developed by the project. These samplers are of three types: depth-integrating, point-integrating, and bed-material.

Depth-Integrating Samplers

Depth-integrating samplers traverse the stream depth at the sampling vertical to within a few inches of the bed, move at a uniform

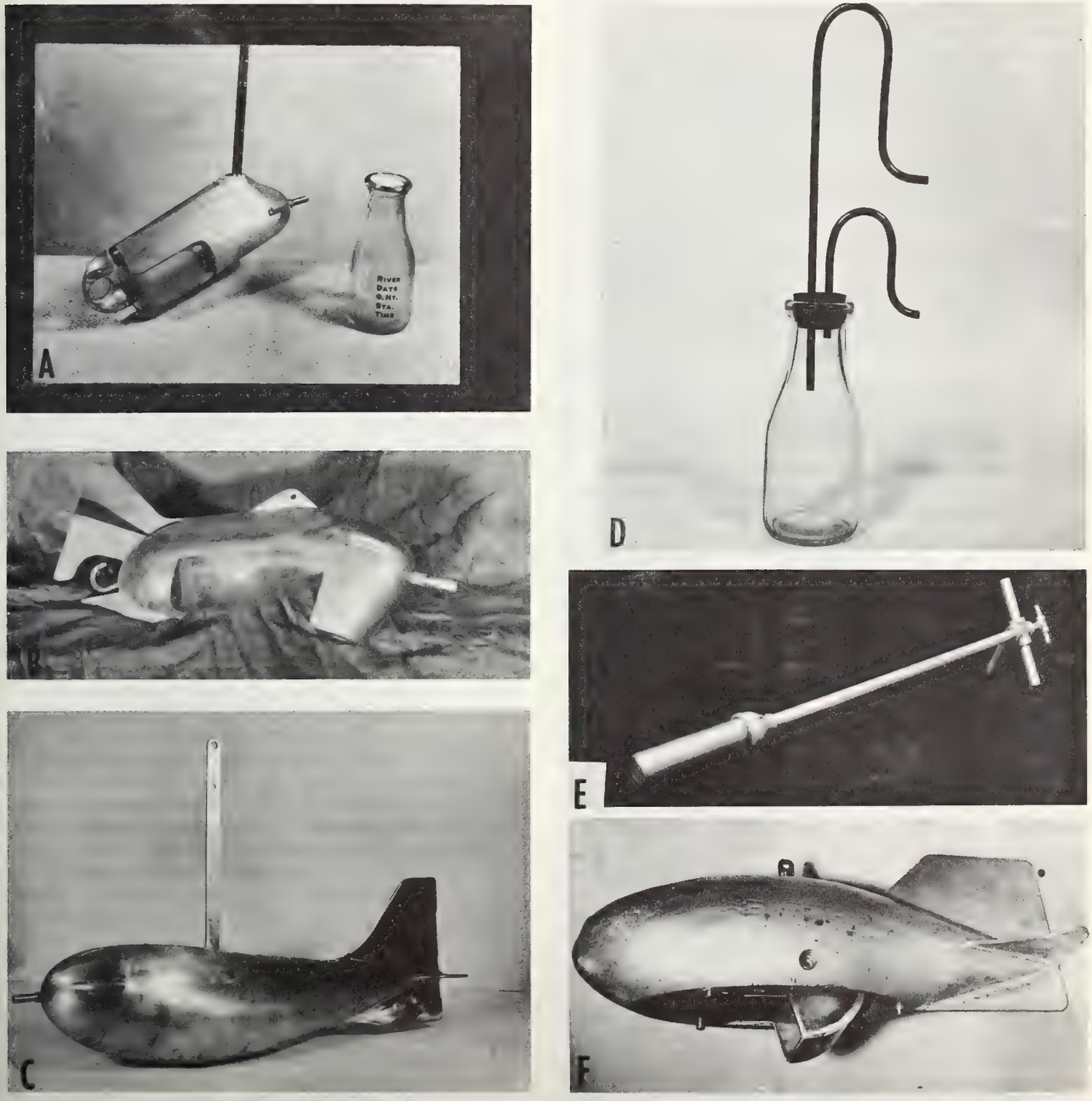


FIGURE 1.— Sediment samplers: *A*, US DH-48; *B*, US DH-59; *C*, US DH-49; *D*, single-stage; *E*, US BMH-53; *F*, US BM-54.

vertical speed, and receive instantaneously at every point a small specimen of the water-sediment mixture, which is later analyzed in the laboratory for sediment concentration and particle size. The nozzle is about 0.3 to 0.4 ft. above the bottom of the sampler. The sample container is a round glass pint milk bottle. An exhaust line allows air to escape from the sample container as the sample enters. The sampler is limited to two-way sampling in depths not greater than 18 ft. One or another of the three types of depth-integrating samplers that have been developed by the project can be used satisfactorily for sampling many streamflows. (The 100-lb. point-integrating sampler is used for depth-integration when depths or velocities are too great for the heaviest depth-integration sampler.)

The US DH–48 hand sampler is used to obtain suspended-sediment samples from streams that can be waded. The sampler is made of cast aluminum and weighs 4½ lb. It partially encloses the sample container. A brass intake nozzle points into the streamflow. A standard ½-inch wading rod is threaded into the top of the sampler body for suspension.

The US DH–59 handline depth-integrating sampler is made of cast bronze and partially encloses the sample container. It weighs about 22 lb. and is equipped with tail vanes to orient the intake nozzle into the approaching flow. This sampler is suspended on a suitable handline.

The US D–49 is a 62-lb. bronze depth-integrating sampler. A hinged head, from which the nozzle projects into the streamflow, permits access to the sample container, which is enclosed in the sampler body. Tail vanes orient the instrument into the streamflow. The sampler is suspended on a steel cable and is lowered and raised by means of a reel mounted on a crane.

Point-Integrating Samplers

Point-integrating samplers collect a water-sediment mixture at one point in the cross-section of a stream. The sample is integrated over the duration of the sampling time. The point-integrating samplers are shaped like the 62-lb. depth-integrating sampler (US D–49, fig. 1) and are equipped with a valve that can be operated electrically after the instrument has been lowered to the sampling point. The valve has either two or three positions: (1) a position for equalizing the air pressure in the bottle with the hydrostatic pressure at the sampling point; (2) the sampling position; and (3) a closed position (in some samplers position 1 is used as a closed position). The samplers are made of cast bronze, and are equipped with tail vanes, a pressure-equalizing chamber, and a hinged head that contains the nozzle, valve, and valve operating mechanism. The hinged head provides access to the sample container, which is a round glass pint milk bottle enclosed in the sampler body. An exhaust port on the side of each sampler head allows air to escape as the sample container fills. The samplers are suspended on a two-conductor steel cable, reel, and crane.

The US P–46–R sampler weighs 100 lb. It can be used for point-integration or to depth integrate on a round trip basis to a depth of 18 ft. or in one direction to a depth of 30 ft. Greater depths can be integrated by dividing the total depth into two or more sections of not more than 30 ft. each. The sampler has a rotary valve that is operated electrically by a rotary solenoid that has the three positions mentioned above. A signal indicates when the valve is in the sampling position. The valve is operated by a telephone dial switch at the observer's station.

A new 100-lb. US P–61 point-integrating sampler is being developed. Two different valve mechanisms have been tested. In the first mechanism a rotary solenoid, when energized, turns a rotary valve from the normally closed and pressure-equalizing position to the sampling position and holds the valve open in the sampling position as long as the solenoid is energized. In the other mechanism, the rotary valve has three positions and the valve is turned by a spring and ratchet. A lever on the side of the head is used to cock the mechanism to the first or pressure-equalizing position, and a solenoid trips the ratchet to let the spring turn the valve to the second and third positions, successively.

The US P–50 sampler is 3 ft. 8 in. long and weighs 300 lb. Either a 1-quart or a 1-pint round glass milk bottle is used as a sample container. An electrically operated slide valve has two positions. The valve is held in the equalizing position by a spring. Solenoids, when electrically energized, hold the valve in the sampling position. This sampler may be used in high velocities and in depths to 200 ft.

Bed-Material Samplers

A bed-material sampler collects a sample of the sediment mixture of which the streambed is composed. It should not be confused with a bedload discharge sampler which samples the rate of discharge of sediment moving as bedload. Only bed-material samplers are described here.

The US BMH–53 is a piston-type hand sampler that is used to collect a sample of sediment from the bed of a stream which can be waded. The sampler consists of a cylinder that is 2 in. in diameter and 8 in. long which can be pressed into the streambed manually. A piston inside the cylinder retracts as the cylinder is pressed into the bed. It helps retain the sample when the sampler is withdrawn and then is used to push the sample from the cylinder. The overall length of the sampler is 46 in.

The US BMH–60 is a 30-lb. sampler, 22 in. long, made of cast aluminum, and used from a handline to collect a sample from the bed of a stream, lake, or reservoir. A cross-curved constant-torque (Negator) spring drives a single bucket that swings out of the sampler body to scoop up and completely surround a sample of about 160 cc. from the top 2 in. of the streambed. When the sampler is supported by the handline and the safety yoke is in place, the bucket can be cocked to the open position in which the bucket is completely retracted into the sampler body. As long as the safety yoke is in place on the hanger, the bucket mechanism cannot be released. When the sampler is supported by the handline and the yoke is removed, a sample can be taken by lowering the sampler onto the streambed until the suspension line is slack. When the sampler rests on the streambed, the mechanism trips and a sample is taken from the bed.

The US BM–54 is a 100-lb. cast iron, bed-material sampler, 22 in. long. When supported by a steel cable, the bucket can be cocked to the open or sampling position. When tension on the cable is released by resting the sampler on the streambed, the mechanism is tripped and the single bucket, powered by a spiral spring, swings out of the bottom of the sampler body and scoops a sample from the top 2 in. of the streambed. The bucket surrounds and encloses the sample in such a way that none of the sediment is washed out.

Automatic Samplers

Two types of so-called automatic suspended-sediment samplers have been designed and tested by the project. One is called the single-stage sampler, and the other is the pumping sampler, which has three methods for handling the sediment samples. These samplers are automatic in that they may be installed at the sampling station and set to collect samples even when unattended.

Single-Stage Sampler

A single-stage sampler is a simple device, which was developed for use on flashy and intermittent streams, at remote or not easily accessible sites where adequate samples cannot be obtained manually.[1] The sampler (fig. 1) consists of a container equipped with intake and exhaust tubes designed to obtain a single suspended-sediment sample automatically when the water surface first rises to a selected stage that submerges the sampler. The sample is collected with respect to gage height and not time.

Samples taken with single-stage samplers are not as representative of the sediment concentration in the stream cross section as are samples taken with manual samplers; therefore, the latter samplers should be given primary consideration at every station. However, the single-stage sampler provides a means of obtaining some suspended-sediment data for streams or storms that would not be sampled otherwise.

A single-stage sampler installation consists of five basic parts:

(1) A sample container such as a 1-pint glass milk bottle;

(2) A siphon-shaped copper air-exhaust tube, with an inside diameter of about $\frac{3}{16}$ in.;

(3) A siphon-shaped copper intake tube and nozzle, with an inside diameter of about $\frac{3}{16}$ in.;

(4) A tight-fitting stopper that seals the bottle and has two holes which hold the tubes tightly in place;

(5) A rack for supporting and protecting one or more samplers one above the other, to sample at different stages.

Single-stage samplers have the following desirable qualities:

(1) Samplers may be installed at the station well in advance of a flood or rising stage;

(2) No one need be present at the time of sampling;

(3) Samples may be obtained at predetermined stages of the stream;

(4) Sampling apparatus is inexpensive.

Limitations of the sampler that may cause errors are:

(1) Samples are collected at or near the stream surface and usually near the edge of the stream or near a pier or abutment;

(2) Size, shape, and orientation of intake and air exhaust elements may fail to provide intake ratios (average velocity in nozzle to stream velocity approaching nozzle) sufficiently close to unity to sample sands accurately;

(3) Obstructions due to the presence of trash and drift during sampling may create unnatural flow lines at the sampler nozzle and cause inaccurate sampling, which may not be detectable later;

(4) Flow may circulate through the sampler after the original sample has been collected.

Four types of single-stage samplers have been designed for different sampling conditions. The vertical-intake type is accurate only for sediments finer than 0.062 mm., for water-surface surges less than 4 in., and for velocities that are reasonably low at the sampling point. Its sampling efficiency usually is little affected by drift and debris, by circulation through the sampler, or by a reasonable amount of shielding if the sediments are fine.

Any one of three horizontal intake nozzles may be used to sample sands as well as fine sediments. The intake nozzles are inclined downward at 20°,

[1] Reference report 13; see p. 177.

15°, or 10° to prevent deposition of sediment in the intake prior to sampling. The smaller angles are used with higher velocities, because at high velocities sediment is less likely to deposit in the nozzle. The height of the siphon-shaped intake and exhaust tubes is selected to accommodate the velocity and surge conditions at the sampling site.

Pumping Samplers

Pumping samplers with three different sample handling systems have been developed and tested.[2] These samplers automatically obtain frequent samples of suspended-sediment concentration of a stream that may not be readily accessible to an observer. Samples are taken from one point in the stream, the point at which the pumping intake is located. Each sampler consists of an intake, pumping system, and sediment handling system.

The intake can be a 1-in. opening in a plate mounted flush on the face of a guide wall that is parallel to the streamflow. The inflow is through a hose to a pump that delivers the sample to a collecting and recording system. A trap in the intake line prevents small fish from reaching and jamming the pump. Any obstruction lodged in the trap is flushed back to the stream just before sampling. The pumping system is operated by a timing device that can be adjusted to take samples at any desired frequency. A safety system shuts off the sampler for 12 hours if the water level falls below the intake or the intake is obstructed.

In the accumulated weight-recording system, samples enter a settling tank where the sediment settles onto a tray. The accumulated weight of sediment is measured by a spring-transformer scale and recorded on a strip chart. The pumping, collection, and recording equipment are housed in an 8- by 10-ft. shelter. The system has a total capacity of 1,300 lb. of sediment and records accumulated weight to the nearest 2 or 3 lb. throughout each of thirteen 100-lb. increments. The average concentration of suspended sediment is computed from the weight of sediment accumulated on the tray and the volume of pumped sample for the measurement period. This sampler obtains a time-weighted concentration that is accurate for fine sands and coarse silts during periods of steady flow; but sudden, or short time, changes in concentration may not be defined.

The sediment volume recording system (fig. 2) consists of a rack that supports 12 sedimentation tubes, each having a constricted section at the bottom for measuring sediment volume; a 16-mm. movie camera for recording the column of sediment and the water level in the sedimentation tube; and a concentric bottle rack that carries 72 pint milk bottles for collecting check samples for analyses in the laboratory. A sample is pumped into one of the sedimentation tubes each half hour. About 5½ hours later, the camera takes a picture of the accumulation of sediment in the constricted section of the tube and also of the height of water in the tube. Then the tube drains in preparation for taking another sample 6 hours after the previous sample was taken. The volume of each sedimentation tube has been calibrated against height of filling so that the volume of the sample and the volume of accumulated sediment can be determined from the picture. The relation of volume ratio to weight concentration is determined for various particle size ranges of sediment encountered at a sampling station. A splitter on top of one of the 12 sedimentation tubes diverts a part of the sample to a pint milk bottle every 6 hours. The milk bottle sample may be analyzed later in the laboratory as a check on the concentration obtained from the pictorial record. This method is not always satisfactory where

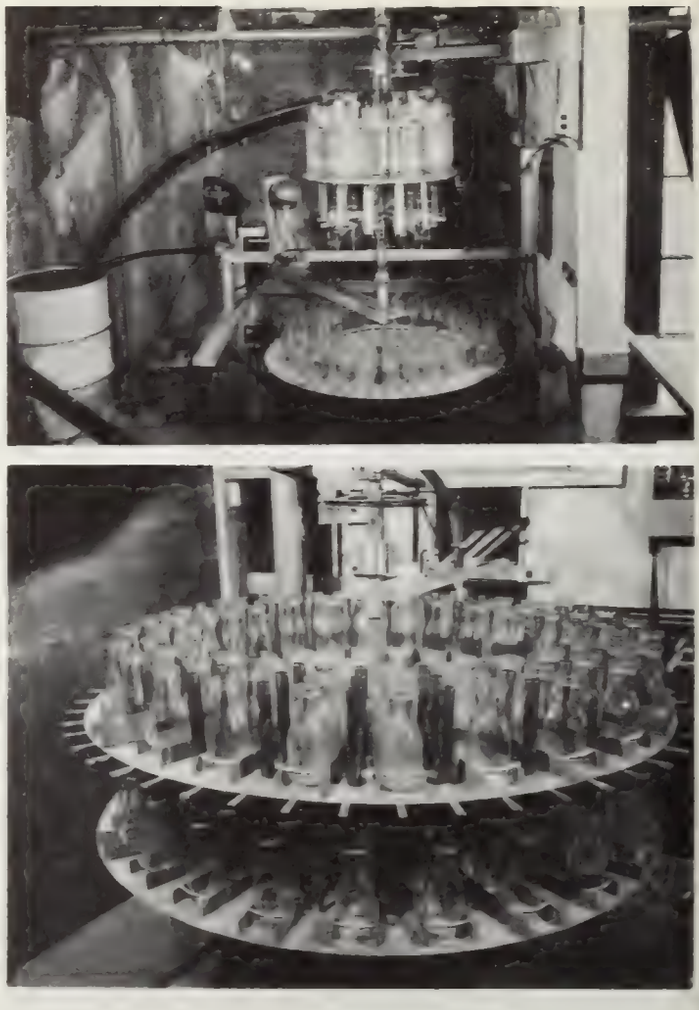


FIGURE 2. — Sample handling systems for pumping sampler: Top, Sediment volume recording system; bottom, individual-sample bottling system.

[2] Reference report Q; see p. 177.

the percentage of clay and silt is high, because the water may be too turbid for a good picture of the water-sediment interface.

The individual-sample bottling system (fig. 2), collects samples in pint milk bottles for analysis in the laboratory. The sample handling system consists of two concentric bottle racks, one above the other, which carry a total of 145 pint milk bottles. Tubes through the upper rack guide the sample to bottles on the lower rack. A special pen records sampling cycles on a river stage recorder. The bottling system can be adjusted to operate for any desired sampling frequency and for any range of river stage by means of a cycling switch and float switch, respectively. This device can collect samples for as long as 72 days without servicing. It is relatively simple and can be made portable.

Sediment Analyzing Devices

Two sediment size analyzers, the bottom-withdrawal tube and visual-accumulation tube, have been developed and four other devices for determining suspended-sediment concentration, or concentration and particle size distribution, are being investigated. The devices include electronic sensing, turbidity, ultrasonic, and nuclear equipment.

Bottom-Withdrawal Tube for Size Analysis of Sediments

The bottom-withdrawal tube was developed to determine size distribution of particles up to 0.7 mm. in samples of suspended sediment.[3] This device is a glass tube 100 cm. long and 1 in. in diameter, with a volumetric scale along its length. The tube is open at the upper end and contracts to ¼ in. at the lower end, which is equipped with a quick-acting outlet valve. The sample is uniformly dispersed in the tube. Then the tube is placed in an upright position and samples of known volume are drawn from the bottom at known time-intervals. The sediment weight in each fraction is determined by drying and weighing. Then the particle size distribution can be computed by use of an Oden curve. The cumulative size-frequency distribution is determined graphically by the intercepts, on the ordinate axis, of tangents to the Oden curve.

Visual-Accumulation Tube for Size Analysis of Sands

The visual-accumulation tube method is a rapid and accurate means for determining the sedimentation-size distribution of the sand in suspended-sediment samples and in streambed and beach material samples.[4] The size analysis is based on a stratified sedimentation system in which the sample is introduced at the top of a transparent settling tube containing distilled water (fig. 3). The sediment accumulates in a contracted section at the bottom of the tube. A manually operated pen is used to trace the height of sediment accumulation on a chart (fig. 4), which moves at a uniform speed.

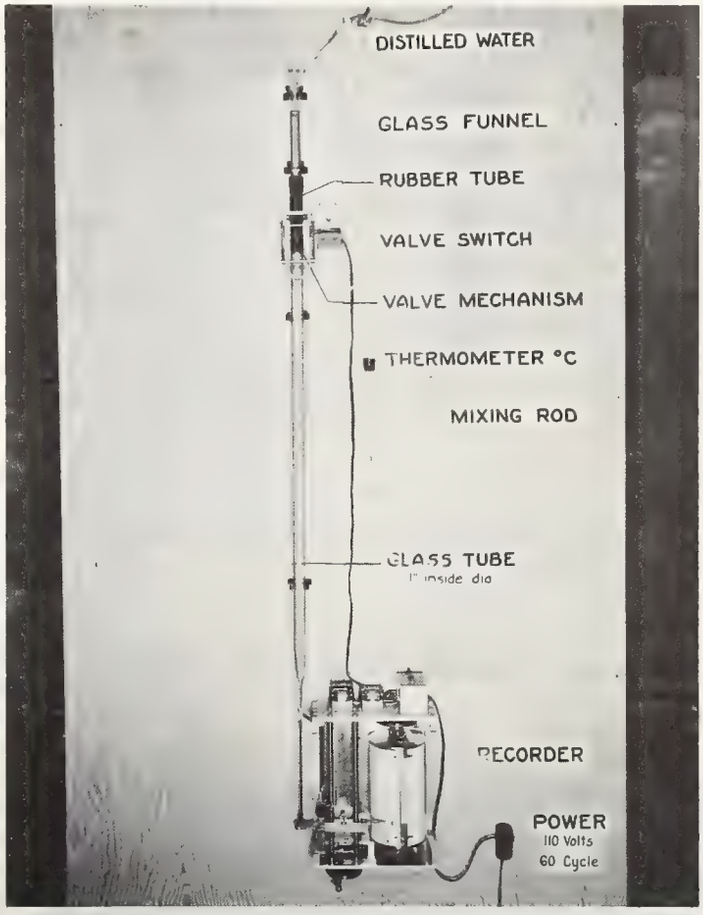


FIGURE 3.—Visual-accumulation-tube sand size analyzer.

The equipment consists of a sedimentation tube and a special recorder. Five sizes of tubes are available. Four tubes have lengths of 120 cm. and inside diameters of 1 in., except for the lower ends that have a constricted sand accumulation section with a diameter of 2.1, 3.4, 5.0, or 7.0 mm. Tubes of these sizes can be used for analysis of samples having a small quantity of sand mostly less than 1 mm. in diameter. A fifth size tube is 180 cm. long and 2 in. inside diameter and has an accumulation section of 10 mm. inside diameter. It is used for analysis of bed and beach or other sands of coarse sizes when sufficient quantities of material are available.

For the visual-accumulation tube method of particle-size analysis the sand portion of a suspended-sediment sample is separated from the silt and clay by sieving or by sedimentation. The 180-cm. tube is satisfactory for analyzing

[3] Reference reports 7 and 10; see p. 176.

[4] Reference report 11; see p. 176.

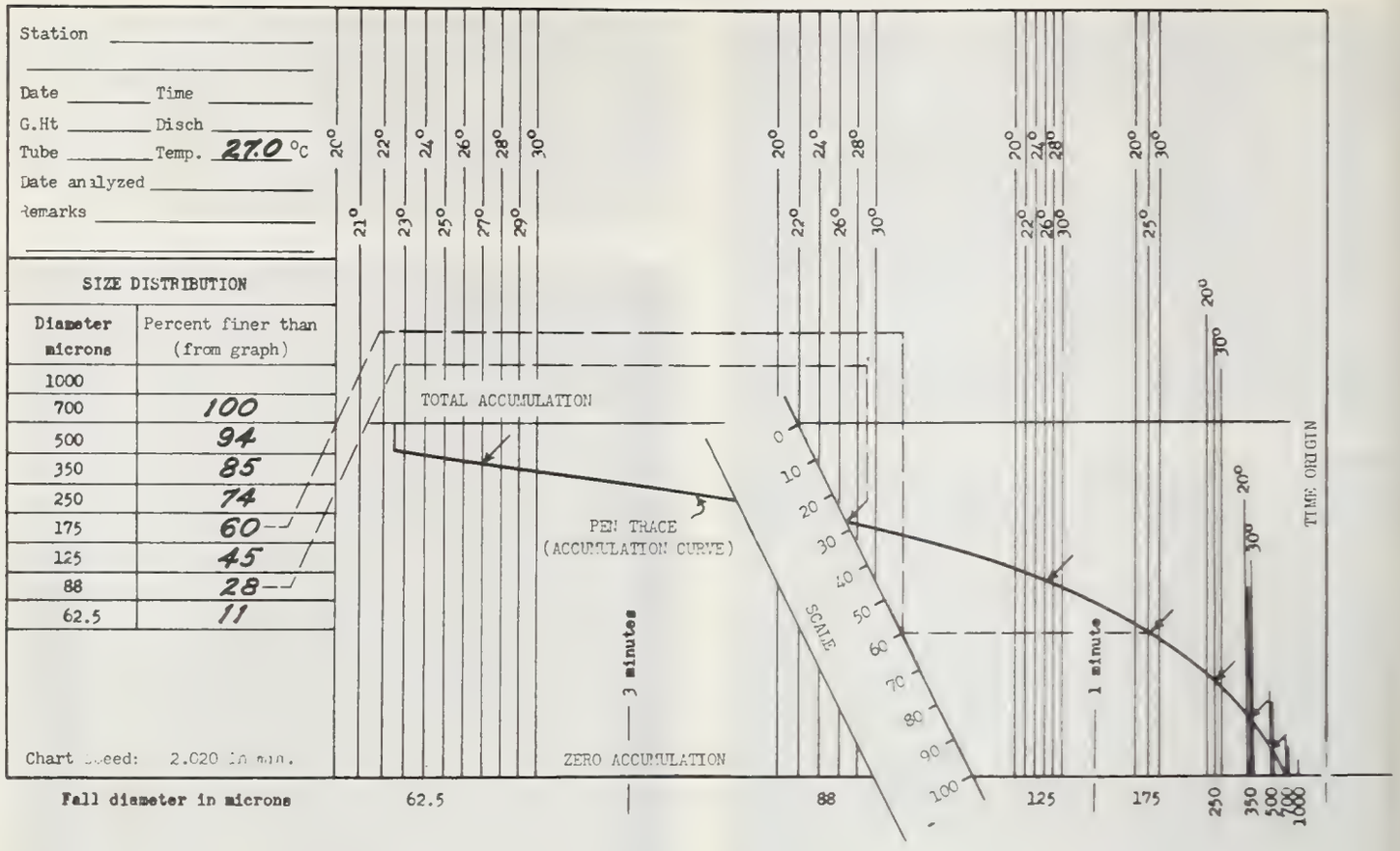


FIGURE 4. — Visual-accumulation-tube sand size analyzer recorder chart for 120 CM tube, showing method of reading size distribution.

samples containing sand up to 2 mm. in diameter. Analyses are in terms of fall velocity of individual particles of which the sample is composed. A sample can be analyzed in less than 10 minutes, which is faster than a sieve analysis. Experience has shown that this sand-size analyzer is a very satisfactory device for determining particle-size distribution for sands in the size range from 0.062 to 2 mm.

Electronic Sensing Device

A commercial electronic sensing device (fig. 5) is being studied to determine its capacity to analyze fluvial suspended sediments for particle-size distribution and sediment concentration.

The counter measures and counts individual particles as they pass through a small aperture. A known volume of an electrolyte (aqueous salt solution) that contains the particles in dilute suspension is drawn through the aperture. The resistance between electrodes on each side of the aperture changes whenever a particle displaces part of the liquid in the aperture. A constant voltage is imposed on the electrodes so that the change in resistance produces an electrical pulse that is proportional to particle volume. The electrical pulses are amplified so that they can be screened as to size and counted.

The counter has an adjustable threshold level below which electrical pulses are not counted. The threshold can be set so high that none or only a few of the largest particles are counted. Then it can be lowered in successive steps and a count of pulses larger than each step can be taken. The relation of pulse height to particle volume can be established by calibration with particles of a known size. Thus, the number of particles in each size range can be determined.

Tests indicate that composition or density of the particles has little or no effect on results of the analysis. The analyzer seems to be an accurate method for determining the number and size of particles in the silt size range.

Turbidity Method

The turbidity method of analyzing sediment is being studied. The equipment (fig. 6) consists of a pump and recirculating system, sedimentation chamber, and the General Electric recording turbidimeter, which contains a turbidity detector and a recorder. Turbidity, which is the cloudiness in a liquid, is measured by a photovoltaic cell in the detector and interpreted in the recorder as the ratio of the light scattered by the particles in the liquid to the light transmitted through the liquid. The transmitted light is measured for 15 sec., the scattered light

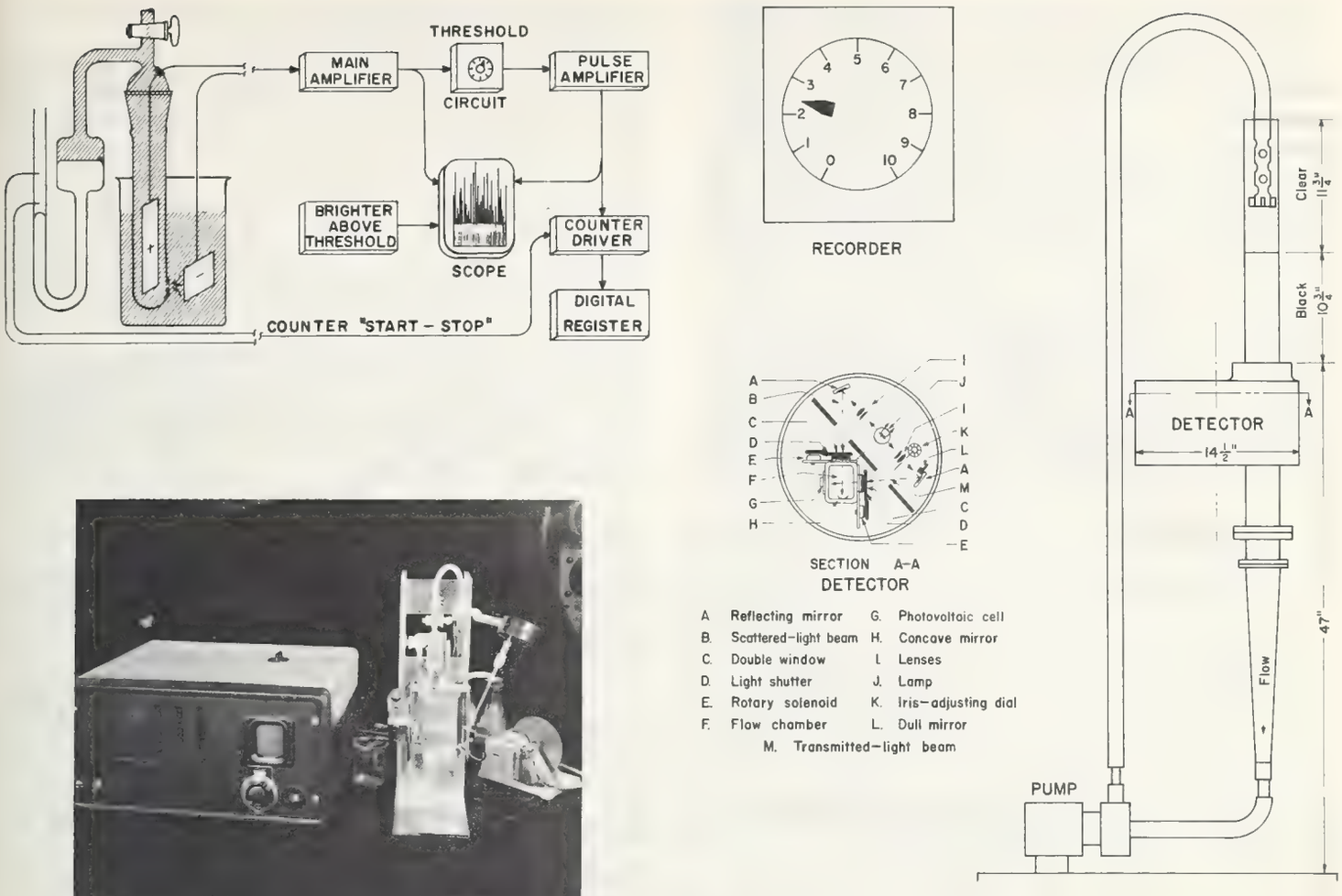


FIGURE 5. — Electronic sensing device for sediment.

for 45 sec., and the ratio is computed and recorded during the latter period. This cycle is repeated continuously. Stability of the ratio measurement is achieved by using a single light source and a single photovoltaic cell.

The turbidity method for determining particle-size distribution and concentration in a suspended-sediment sample consists of two steps: (1) Turbidity of a sample being pumped through the detector is recorded continuously for determination of concentration; (2) for particle-size determination, the flow is suddenly stopped and turbidity is recorded against time as particles settle out of suspension. Particles of uniform density and shape fall at rates proportional to their size, and light is obscured in proportion to particle size and concentration. The relative weight of each particle-size group required to obscure light to a given degree is represented by a hiding factor. The weight of sediment in each size group is computed in percentage of the total for each incremental change in turbidity by using the equivalent hiding factor and making adjustment for concentration.

Calibration tests of this method were made to

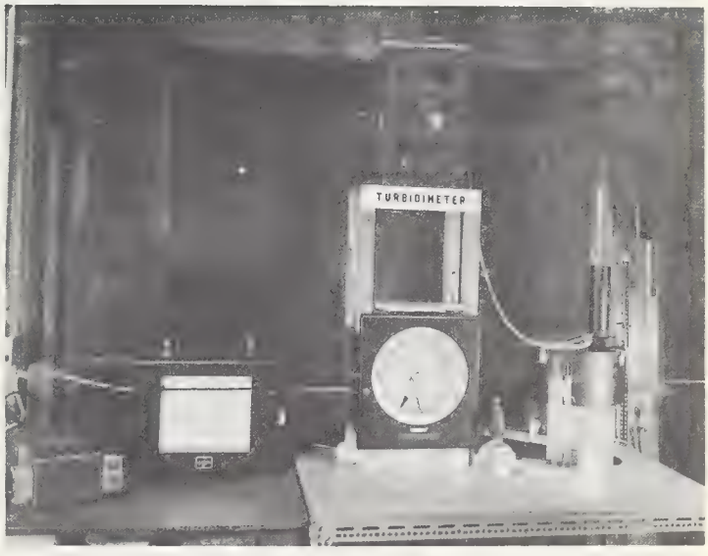


FIGURE 6. — Turbidimeter to analyze sediment.

determine the relation of turbidity to the characteristics of various types of sediment. Observed turbidity and settling time were related graphically for each tested concentration. Particle size was computed by Stokes' Law for settling time and temperature observed in the tests.

Then particle size was related to turbidity graphically through common values of settling time. Size distribution was computed as cumulative percentage of each size fraction and was presented by a log-probability plot on which cumulative percentage finer is plotted against size. Particle-size distribution is defined by computing, from the log-probability graph, the geometric mean diameter and the geometric standard deviation.

After particle size has been determined, concentration is found from a calibration graph (fig. 7) in which turbidity is plotted with respect to concentration and the geometric mean diameter and the geometric standard deviation are used as additional parameters. A graph such as that of figure 7 is prepared from tests on known samples of the particular type of sediment that is to be analyzed.

The turbidimeter can be adjusted to measure fairly accurately a wide range of concentrations, from as small as 0 to 10 p.p.m. to as large as 0 to 100,000 p.p.m. Also, it can measure the sizes of particles within the approximate range from 0.020 to 0.120 mm. Finer particles settle so slowly that a test may take an excessively long time, whereas coarser particles settle so rapidly that an unreasonably tall sedimentation chamber may be required.

The principle of the turbidimeter appears to be adaptable to a field instrument for monitoring concentrations and particle-size distributions of suspended sediment pumped continuously from a stream.

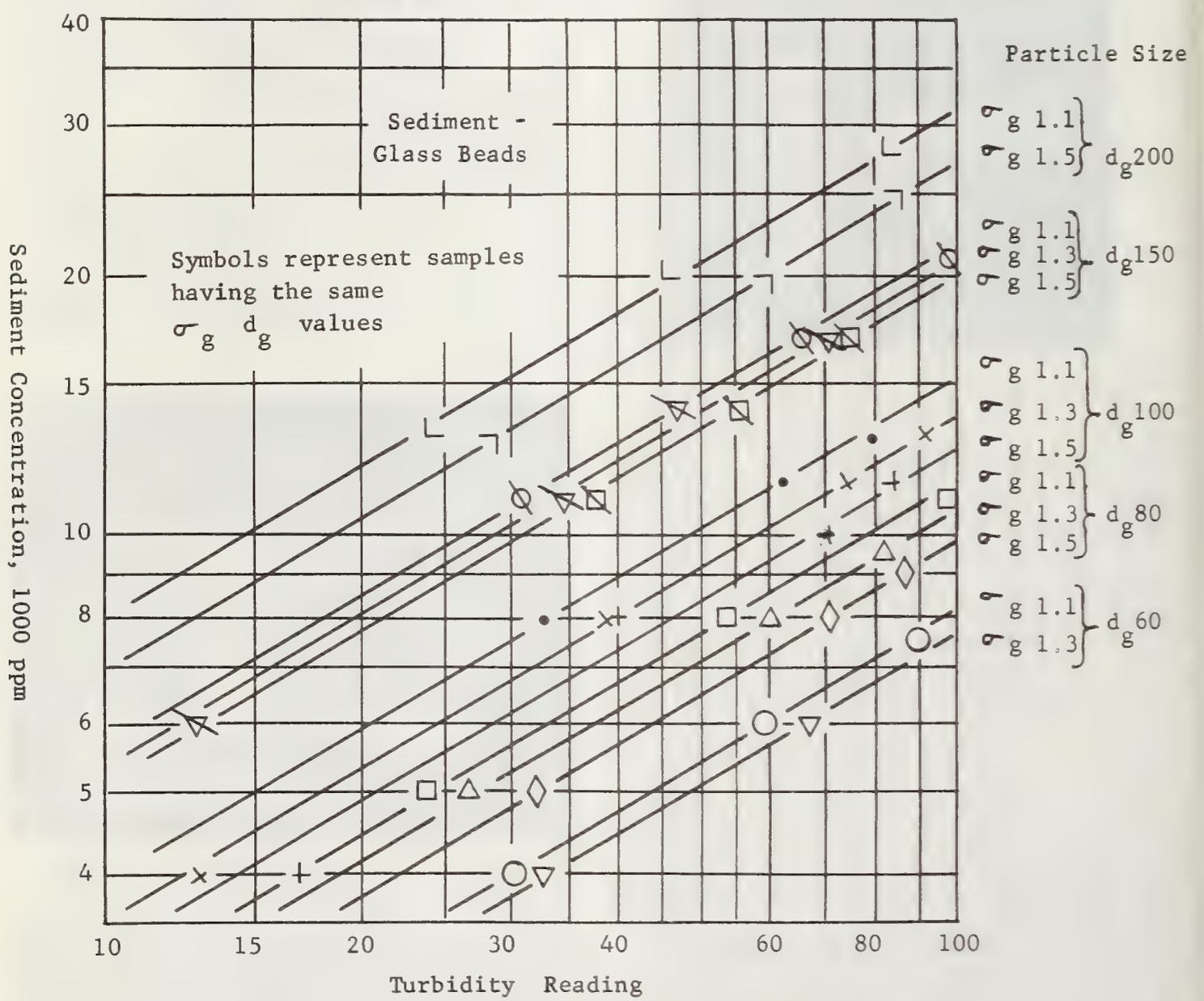


FIGURE 7. — Concentration-turbidity calibration graph.

The results of a particle-size analysis (fig. 8) of a prepared sample of glass beads were about the same whether made with the electronic sensing device, the turbidimeter, or the visual-accumulation tube.

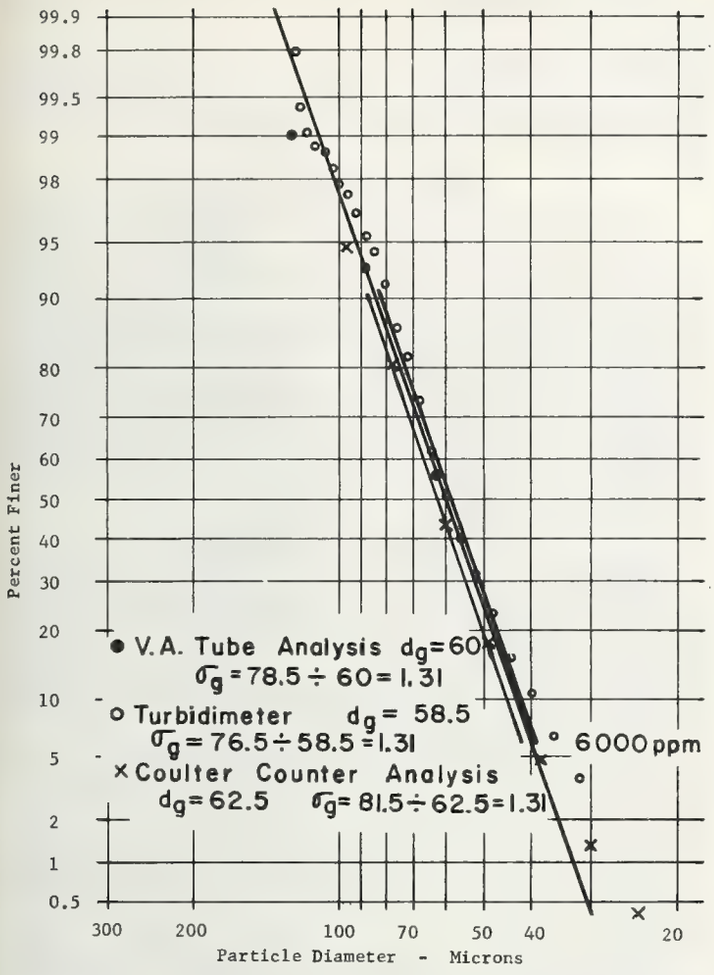
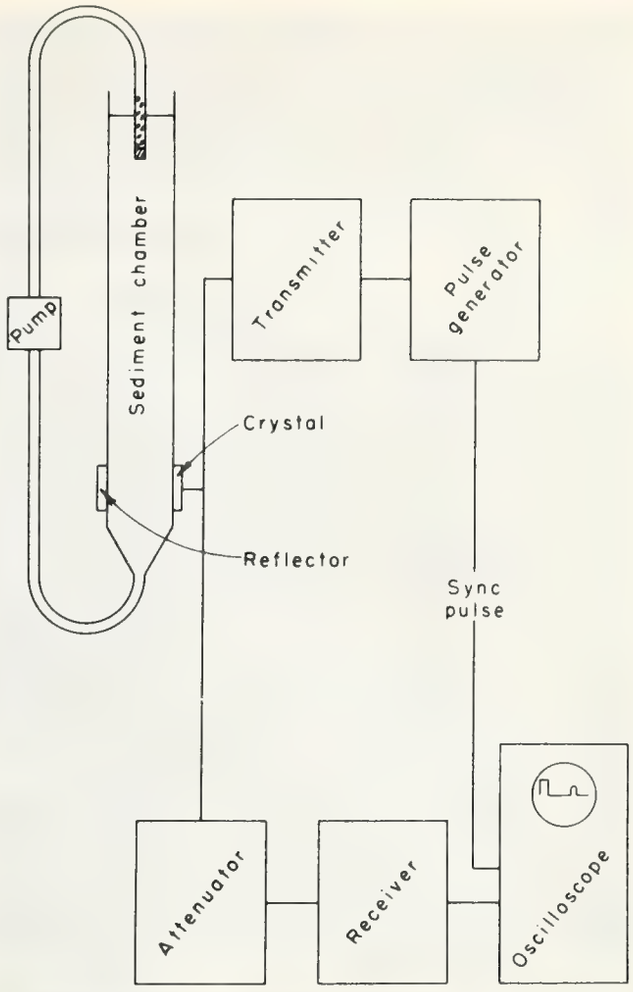


FIGURE 8. — Results of visual-accumulation-tube, turbidimeter, and electronic sensing device.

Ultrasonic Method

Ultrasonic equipment is being developed to determine size distribution and concentration of sediments in sizes from 0.040 to 1.0 mm.

In this equipment (fig. 9) a high frequency electrical current is imposed on a quartz crystal for about 6 micro-seconds at a repetitious rate of 200 times a second. The quartz crystal, which is mounted in the wall of a sedimentation tube, converts the electrical pulses into displacement pulses or sound waves. The wave passes through the fluid in the sediment chamber and strikes a second crystal in the wall on the other side. The two crystals are of matched frequencies. The second crystal acts as a transducer and converts the sound wave to a secondary electrical current. The electrical current is fed back through an attenuator and displayed on an oscilloscope.



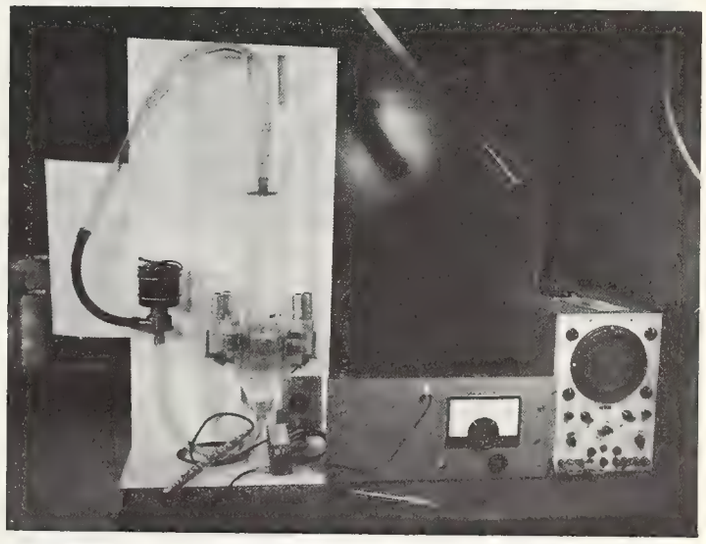


FIGURE 9. — Ultrasonic equipment to measure sediment.

The effect of sediment on attenuation of the sound wave is determined as follows: With water in the sediment chamber, the top of the oscilloscope trace is adjusted to a reference line on the oscilloscope screen. Then sediment is introduced into, and circulated through, the sedimentation chamber. The presence of the

sediment reduces the height of the trace. Electrical resistance is removed from the attenuator in the secondary electrical circuit until the trace is restored to the original height on the oscilloscope. The electrical resistance that was removed is a direct measure of the energy loss caused by the addition of the sediment. The energy loss varies directly as the concentration of sediment.

Unfortunately, attenuation of ultrasonic energy depends as much on particle size as on concentration. Although the ultrasonic equipment can be calibrated to determine quickly and accurately the concentration of sediment of a known single size, or a single size distribution, the determination of the concentration of sediment of an unknown size distribution is difficult. For sediments that have a size distribution that can be expressed by two parameters such as the geometric mean diameter and the standard deviation, it is possible to determine the size distribution in an unknown sample. Calibration and analysis must cover a range of about 10 frequencies. After the size distribution is determined, the concentration can be found.

The ultrasonic equipment operates reasonably well in the laboratory, although further development will be needed before it will be competitive with more common laboratory methods of analysis. The potential for field use has not been evaluated.

Nuclear Approach to Analysis of Sediment

The possibility of using radioisotopes for determining the concentration of suspended sediments is being investigated. The Atomic Energy Commission and Parametrics, Incorporated, are cooperating with the Inter-Agency Sedimentation Project on a study of nuclear possibilities.

The first phase of the investigation is a study by Parametrics, Incorporated, of the feasibility of using X-ray emitting radioisotopes for the measurement of the solid-liquid ratio in natural waters. The investigation is both theoretical and experimental with emphasis on the following items:

(1) Chemical constituency and variation of sediment and flow.

(2) Investigation to determine whether X-ray, gamma, or beta emitters are best.

(3) After selection of a general type of emitter, determination of availability, cost, and licensing potential.

(4) If a single isotope is strongly suggested from item 3, further study of this isotope only. Otherwise, the best two isotopes will be considered for the remaining parts of the investigation. If the multisource method seems best, several isotopes will be considered.

(5) Prediction of detector response, accuracy in determining percentage of solids, and source strength requirements as a function of liquid density and concentration of solids.

(6) Basic confirmation of predictions with limited laboratory experimentation. Modification or sophistication of response theory will be made as indicated.

(7) A preliminary study of the suitability of various detectors as a basis for recommendations.

(8) A preliminary investigation of circuitry techniques suitable for data processing to obtain solids concentration automatically.

Future Program

Project efforts in the near future will emphasize (1) preparation of an up-to-date report on the measurement of fluvial sediment discharge, (2) field tests of the pumping samplers, and (3) development work on the electronic sensing, turbidity, ultrasonic, and nuclear devices for analysis of suspended sediment samples in the laboratory and for automatic collection and analysis of suspended sediments in flowing streams.

List of Project Reports

Reports on the cooperative study of methods used in measurement and analysis of sediment loads in streams covers phases indicated by the following titles prepared by the Inter-Agency Sedimentation Project, St. Anthony Falls Hydraulic Laboratory, Minneapolis:

Report No. 1. — Field practice and equipment used in sampling suspended sediment, August 1940.

Report No. 2. — Equipment used for sampling bed load and bed material, September 1940.

Report No. 3. — Analytical study of methods of sampling suspended sediment, November 1941.

Report No. 4. — Methods of analyzing sediment samples, November 1941.

Report No. 5. — Laboratory investigations of suspended-sediment samplers, December 1941.

Report No. 6. — The design of improved types of suspended-sediment samplers, May 1952.

Report No. 7. — A study of new methods for size analysis of suspended-sediment samples, June 1943.

Report No. 8. — Measurement of the sediment discharge of streams, March 1940.

Report No. 9. — Density of sediments deposited in reservoirs, November 1943.

Report No. 10. — Accuracy of sediment size analyses made by the bottom-withdrawal-tube method, April 1953.

Report No. 11. — The development and calibration of the visual-accumulation tube, 1957.

Report No. 12. — Some fundamentals of particle-size analysis, December 1957.

Report No. 13. — Single-stage sampler for suspended sediment, 1961.

Report AA. — Federal Inter-Agency Sedimentation instruments and reports, May 1959.

Report Q. — Investigation of a pumping sampler with alternate suspended-sediment handling systems, progress report, June 1962.

Note. Other lettered reports, A to P, are manuals on sediment sampler operation or studies of sampler and equipment operation.

DISCUSSION — SYMPOSIUM 1
Land Erosion and Control

Moderator: John B. Stall, Illinois Water Survey

Panel: Russell Woodburn, ARS
Sam Maner, SCS
Stan Ursic, FS

MR. STALL:

Gentlemen, we have been treated today to 25 papers on Land Erosion and Control. I like to think of these as being building blocks or bricks as mentioned yesterday by Dr. Vanoni. Underlying them all is a structure that governs sediment movement and which we are all seeking. After hearing these papers, we should reflect to some degree as to how these may fit together; as an attempt to do that in panel form, we have asked three persons to do this who are willing to make public for you their reflections as to today's activities. These people are Sam Maner of the Soil Conservation Service, Stan Ursic of the Forest Service, and Russell Woodburn of the Agriculture Research Service.

We have a number of questions from the question box for the individual speakers. We will ask them to answer these. We hope, however, that the three brief summaries that these men are going to give now may stimulate your own thinking somewhat, either by agreement or disagreement, and move you to participate in the discussion which will follow. To begin with, I want to call on Mr. Maner. I have told all these men they can be as brief or as long as they like up to a couple of minutes.

MR. MANER:

Since Mr. Bryant introduced those statisticians to the conference yesterday, I would like to place our kind of work — which is more or less a very small flood control or flood prevention type of structural program — in the same light as compared to some of the things that you people can do in the Bureau of Reclamation and the Corps of Engineers. When those two people missed the target in that joke yesterday, 3 feet on one side and 3 feet on the other and concluded that between them they got a dead center deal, we have to get that dead center more or less on our little structures because we do not have the leeway that you people may have. (sic) In other words, in some of your large structures out here you might be able to get by with an area of a foot of sediment, whereas a foot or 6 inches is all we have to work with. So we are the dead center group when it comes to that. My general impression is that this whole deal, so far, has been very good and that is what you would expect a guy to say when there are 270 people out there who might disagree with you if you said it was bad. Seriously, it is real good.

One of the things that was most gratifying yesterday was something Dr. Wolman said. I, myself, feel like when you classify these people, most of those in the Soil Conservation Service doing sedimentation work are supposed to be geologists. I think in the other agencies a big portion of these people might be hydraulic engineers. Being classified as a soil conservationist, I am somewhere between the geologist and the engineer. Dr. Wolman said that, strictly speaking, there is some place where the geologist quits and a point where the engineer begins, and I hope I am somewhere in between those two guys. I don't believe I will take any more of your time. If you have a question as far as my paper is concerned or on the type of work I have been doing, which is primarily sediment delivery ratio studies as they relate to the physical characteristics of watersheds, I will be glad to answer those either now or after the discussion.

MR. STALL:

Next, I want to call on Stan Ursic of the Forest Service.

MR. URSIC:

John, this assignment reminds me of Otis Copeland's story of the sailor shipwrecked with 12 beautiful girls. Perhaps it's more like the sailor on a weekend pass who has a larger array of material to sample but who is handicapped by the time factor and certainly by other considerations.

Vegetative cover prevents accelerated erosion and sedimentation. This salient fact was stressed by a great majority of the papers presented here today. The problem is one of economic use of the land to produce the food, the fiber, and the wood products we need.

One solution has been suggested — move all

the people off the land and let nature take over again. Perhaps in view of recent world events, this possibility is becoming a little less remote.

Forests — The major problem on forest lands is caused by disturbances. Other than the catastrophic effects of wildfire, the principal disturbances are those associated with logging activities. However, with proper management practices, as brought out by the papers of Howard Lull and Dick Fredericksen, sedimentation as measured by stream turbidity can be reduced to prelogging levels within a period of 2 or 3 years. Perhaps some of you may have questions as to whether or not this applies to areas with less rainfall.

Range — The problem is one of proper utilization, and here two papers have significantly pointed out that very small changes in vegetative cover are important. These are the papers of Mr. Lusby and Mr. Otis Copeland.

Open Land — Here, I believe, the problem is one of minimizing the time when the soil is left exposed to rainfall energy and the application of measures designed to reduce movement to channels.

Of the 25 papers presented, 14 dealt with sedimentation sources, 6 with erosion control, and 5 with miscellaneous topics.

The sources are not so much a problem for research as they are a problem in management. As Mr. Bullard pointed out, we have the know-how; it boils down to the question of how much we are willing to pay to take care of them.

Control — None of the papers have given any indication that erosion problems cannot be solved. On the contrary, there is an undercurrent of optimism, and I believe rightly so. As noted yesterday, none of the papers have dealt with the present extent of erosion. How much work has been done in assessing the erosion problems in this country? Are these data available? How much more work is needed? Are these data available? Lastly, in the papers presented, there has been no suggestion of new or drastic or imaginative methods to control erosion, except perhaps that of leveling the hills to reduce slopes. Thank you.

MR. STALL:

Thank you, Stan. Next, I want to call on Russell Woodburn of Agricultural Research Service.

MR. WOODBURN:

Gentlemen, I would say this is a fairly easy assignment. It is about like giving a man the job of summarizing the Sears, Roebuck catalog in 5 minutes. I have been impressed with the way that the program began this morning with Mr. Roehl's discussion of soil erosion and its control. He started his discussion of sedimentation with erosion which, it is very likely, is the beginning of the whole process. He followed that through the various aspects of erosion — upland erosion, sheet erosion, small channel, then the extreme case of gully erosion. That theme has been carried on further by other speakers throughout the day. Then we moved on down into flood plains. Mr. Parsons and some of the other speakers mentioned channel problems, another special case of erosion, channel instability, and the protection of channels. Then we moved on into some of the areas covered by other speakers — watershed characteristics and the relation of these various characteristics to erosion and to sediment production.

The general difference was touched upon, at least to some extent, of erosion and sediment production as such. We have covered then a very broad spectrum of the problem of erosion and some of the remedial measures for erosion and the relationship to sedimentation. I was impressed with the fact mentioned by several speakers that a little bit of grass goes a long way in solving this problem and by that, Mr. Ursic, I would not leave out the trees. To me, one of the most impressive parts of this whole discussion has been the common denominator of the erosion problem and the sedimentation problem faced by all of our Federal agencies dealing with water in its broad aspect. We all have to face the problem of erosion and the problem of sedimentation. It is gratifying to know that progress is being made as reported by a number of the speakers and the progress in general has consisted of the application of a lot of hard work and the application of many of the old well known and well established principles, and, thus far, I have not seen nor have I heard reported any amazing break-throughs or any miracle drugs to solve the disease. Thank you.

MR. STALL:

Thank you, Russell. In my own case, my principal reaction here today has been — what a sheltered life we lead up in Illinois in the Cornbelt! A reasonable rate of sheet erosion from our watersheds might be 4 tons per acre per year and a reasonable rate of sediment in the reservoirs may be equivalent to 1 ton per acre per year and some of the figures we have seen here today just astound me. Now, we have 10 or 12 questions which I am going to take in order. I hope the authors in question are still here. We will allow a limited contribution from the floor or question from the floor if it is associated with the question or point we have under discussion at the moment.

The first is a question of Mr. Beer who talked about gullies in Iowa and the question is this: "In explaining gully growth, Mr. Beer, the

equation contained a factor linked from the end of the gully to the watershed divide. Would you explain the coefficient and the exponent of this factor? What is the reason for the importance of this factor? And does the gully area increase at a geometrical rate as the gully progresses upstream?"

MR. BEER:

First of all, to give some thinking on the inclusion of the X_{14} which is the variable in question. You noted from Mr. Jacobson's first slide and the first slide that I had that you must, I believe, include some factor that measures the potential for widening along the existing gully. That was the reason for X_8 for the length of the present gully. In other words, any time you get a little bit of concentration of water along the edge of the gully, you are going to get widening. Now, with that, the idea was that you must then provide some measurement for the amount of runoff contributing to the overall or lengthening of the gully. That was the reason for the inclusion of the X_{14}. You might ask why do you use length instead of areas. Well, it is a simple fact — we were not thinking far enough ahead to have those areas delineated whereby it could be measured, but I am not too much concerned, because the analysis showed approximately 0.85 correlation between the watershed length and the watershed areas, so I think it is at least an index. As to the coefficient, I do not think I am in a position at this stage to say whether it is right or wrong.

Surely, the validity will have to be established with further sampling, and that, of course, is what we plan to do. We will fly Steer Creek again this year. We plan to add a second watershed adjacent to Steer Creek. What I will do is take these data, feed it back into the equation in hand, and then compare the predicted values and see how we are coming along on that. As for the last part of the equation — does it increase in geometric ratio — I don't think I am in a position to answer that yes or no, as I made no correlations between area change and headward growth.

MR. STALL:

Thank you. Does anyone have any comment? The next question is to Mr. Striffler. "The implication of the paper is that land use has affected trout streams. The conclusions indicate that bank erosion is a primary determinant of sediment load. How has land use influenced bank erosion and also how was the bank erosion measured?" Mr. Striffler?

MR. STRIFFLER:

Well, let me answer the first part of that. How did land use affect bank erosion — is that the question?

MR. STALL:

Yes.

MR. STRIFFLER:

It is difficult to point to any specific relationship between land use and the amount of bank erosion within any one watershed. However, we can say several things about bank erosion. In this study a large proportion of our eroding banks occurred where stream bottom lands are pastured. I think this was brought out in the paper. In Michigan we also have another situation where we have very high eroding banks. These may occur in our forested areas as well as in our agricultural or pastured areas. These high banks have been attributed to several causes. First of all, some people say that these banks were originally started during logging days when large numbers of Michigan white pine logs were rolled down these banks into our streams. I cannot accept this as being the primary cause of these high banks. I think that several other things are more important; for instance, reservoir construction downstream which has influenced the grading of the stream, has started a cycle of erosion, which has progressed up these streams. I don't know if that answers that question satisfactorily or not.

MR. STALL:

How did you measure bank erosion?

MR. STRIFFLER:

The bank erosion variable used in the analysis was the total length of eroding banks within each sample watershed. These were measured by the Fish Division of the Michigan Conservation Department, and this information is available in their survey and plan reports.

They actually conducted a field survey where they had a team walk the stream bank and measure the length of each eroding bank. Where their survey did not cover the part of the area I was working with, we did our own survey.

MR. STALL:

Thank you. Any further comment or question? I next have a double-barreled question. I believe I will ask Mr. Betson to answer the first one. "Several of today's papers have reported on complex curve fittings and multiple correlation analyses, with many variable and constraints, using computers. Although the computer may correctly solve the mathematical problem given to it, what assurance is there that the formulation of the problem and interpretation of the mathematical results are correct?" Mr. Betson?

MR. BETSON:

The reticence to use the computer kind of reminds me of a quotation which is not original

with me but goes like this: "Even though I don't understand the digestive system, I continue to eat." Proof must lie with the user. Mathematical models are set up usually based on the mass of knowledge we are able to accumulate at the time. If this is fitted to data, the answers will either verify the theory or they will not. If they verify the theory, we are in good shape. What we thought all along is true. If this is not the case, one of two things can be wrong. Either the theory is wrong or the data are wrong. Let's assume the theory is wrong and this has happened. In this program I talked about, we have used it to fit data and found that our theory was a little bit shoddy. We have had to revise our thinking and in light of the revised thinking, we have found out a lot more about watershed characteristics than we knew beforehand.

The other possibility is that the data are in error. Again, we are getting back to the point where the hydrologist or user has to have enough understanding of what he is doing to be able to detect when this happens. The computer techniques are, in a sense, not much different than any other techniques. We can analyze data to death but in the end result, it either gives us the answer we are looking for or it doesn't. It is up to the user himself, I think, to prove whether the answers are consistent with the thinking or not. Does this answer the question?

MR. STALL:

In further answer to this question, I would like to comment myself that it is my feeling that the use of the computer or mathematical model can never relieve us of our own responsibility for understanding of the phenomena involved. The researcher must develop a pattern of understanding into which he can fit his observations somewhere. Then if the result is a complex model to solve, you can take advantage of some of the nice things that have been done in the way of curve fitting as described by Mr. Betson. I feel that the use of mathematical models in an attempt to bare this structure of understanding that Dr. Vanoni talked about is very promising. We should make attempts to reveal this basic structure; but in doing so, usually we cannot ask our own set of observation to give us all the answers. We are required to construct this model in accord with known physical laws and well-substantiated theories inferred from other observations. We then ask the computer to solve this model as we set it up. These complex models are a great help when your own thinking leads you to believe that certain factors should be included in a complex manner. You can often solve it by a mathematical model, but you are still called upon to formulate the problem yourself.

Now, the second question has to do with somewhat the same subject. "What is the effect of errors in the data on the result of multiple correlation analysis. Can small errors lead to large changes in fitted formulas when all the correlations are relatively weak?"

MR. BETSON:

Yes. I think that this is true, and I believe the person asking the question has a good point. This is, of course, one of the many limitations of the multiple regression. It is inherent. Again we have to recognize it, and this regression can give us no better answers than the data we feed into it.

MR. STALL:

I know there are a lot of people here who have experience using multiple regression and some of the more refined probability methods, and I am wondering if anyone would like to comment further on it? Al Sharp? Mr. Sharp has studied this quite considerably — the multiple regression and its uses in studying water yields in the Great Plains.

MR. SHARP:

Thanks, John. I would just like to raise a red flag or two, in use of multiple regression and other statistical treatments of hydrologic and other similar data. First of all, if one has a great many variables and comes out with a great many equations and a great many correlation coefficients, etc., one is very likely to get spurious results because, just by pure chance, one out of every hundred equations may show up highly significant, or five out of every hundred may show up significant, when actually there is no real significance. So, I think all our analyses should bear up under very, very rigid logical examination.

Another remark I would like to make is that I question very much the application of much of our data to the multiple regression technique, because it (the multiple regression technique) was derived from one kind of data and we are applying it to another, and our data just don't fit the premises upon which the multiple regression method is based. I won't go into this; any of you can do that.

Third, I have been trying for years and years to promote a study or project and I want to make an appeal for some support. We do not have statistical procedures now available that properly fit our hydrologic and sedimentation data in many cases. I think it would be highly desirable if, among us, we could derive some support or build up some pressure for such a study. I don't know whether anything could

come of it or not, but we certainly ought to know. Every one of us uses a double mass diagram. Nobody knows how much of a break in such curves is necessary for the change in slope to be significant or due to chance. There are many other similar problems involved, so I think that we as sedimentationists and we as hydrologists ought to develop our own statistical procedures through, perhaps, some joint effort.

MR. STALL:

I know there are number of others who have talked about this subject and would like to comment, but let us go ahead. The next question is to Henry Anderson. "Would you please comment on the effect of water temperatures on sediment yield?"

MR. ANDERSON:

I suppose the answer to that is that we had no measures of water temperature associated with our sediment samples. Some of these people have made the correlation on the effect of temperature on sediment yields, which seemed quite startling. We have made none directly on that. We have made evaluation of the relationship of temperature of the area to soil development, and some of the equations in the paper bear on this problem. We have been using the dissociation constant of water as an expression of the temperature of the climate of the region in developing soils. Very highly significant developments have come from this. Is this what this person had in mind?

MR. STALL:

I believe that the implication is just what you have commented — that the effect of temperature can be very enormous, if evaluated, and that if you had information on temperature in your study, we would like to know about it. Apparently you do not.

MR. ANDERSON:

We have measured no measures of temperature of the water associated with sediment measurements. However, in some analyses you do this on a month by month basis and then, of course, this is dealt into the thing in effect.

MR. STALL:

The monthly variation of temperature is inherent in your study. Next, we have a question to Mr. Chepil. "Have you found any significant evidence that saltation and/or avalanching has contributed significantly to bedloads in streams?" While we are waiting for Mr. Chepil, I guess all of you here could not help but notice the similarity between the movies Dr. Chepil showed and those I have seen of Dr. Einstein's — similar movement of bed particles along the bed of the stream. When the wind picks these particles up, they tend to rise vertically and carry farther and tend to be far more violent movement. There is a very striking similarity, and that is the area of this question.

DR. CHEPIL:

We have no measurements of the relation of saltation to bedload movement in streams, and I can't answer that question.

MR. STALL:

Don Parsons has a question.

DR. CHEPIL:

The question is, "What were the approach conditions for the transport of grains as shown in the movies?" I construe this to mean under what conditions the photographs were taken. The length was 4 inches, and the height was 2½ inches. The area was illuminated by a beam of light, which was shot from the top about 1 cm. wide. Concentrated sunlight was used about three times the intensity of sunlight. So what you saw was only the particles within that vertical beam of light, which was parallel with the direction of the wind. Does that answer the question?

MR. STALL:

We have a question from the floor.

MR. NORMAN BROOKS:

How much channel is there upstream?

DR. CHEPIL:

About 5 feet of erodible soil surface. That was sufficient to cause particles to jump at least a foot in height. Many particles jumped much higher than that; about 90 percent of the particles jumped within the height of 1 foot. The rest jumped above 1 foot, or higher. These particles were in saltation up to about ½ mm. in diameter. The length of the upstream air channel was about 50 feet. The length was sufficient for development of an air boundary surface at least 1 foot deep.

MR. STALL:

What was the wind velocity?

DR. CHEPIL:

The wind velocity was about 25 to 30 miles per hour at about 1 foot height. That is equivalent to about 40 to 45 miles per hour at 50 foot height.

MR. STALL:

Thank you very much, Dr. Chepil. We have a question now to Mr. Piest. "Did you take into account the effect of very high floods in permanently changing the watershed, such as by starting gully systems, by changing river channels permanently?"

MR. PIEST:

I don't know whether I understand that question correctly or not. Did I take into account very high floods?

MR. STALL:

Yes — which, by their extreme nature, might originate an entire gully system or channel system?

MR. PIEST:

Let me say that most of the records that I was dealing with were 3 to 7 years in length; few of them were a little bit longer than that. Most of the time you won't have an extreme event (like a 50-year storm) in that short-length period, although with the extrapolation procedure you are supposed to take account of that. However, in several instances there did occur, in the short-term periods, storms that had return periods of 50 years or more. One of these was in Ralph Baird's watersheds at Riesel, Tex. These were 176-acre and 132-acre watersheds, and there was no channel alteration that I know of. One of the drawbacks in my analysis of the data is that, in a lot of cases, I was unacquainted with the particular area. The extrapolation procedures on the water-sediment relationship were, I think, pretty good. However, because there was no long-term stream gaging data in the immediate vicinity of the subject station, we would have to go, sometimes, 25 to 50 miles away for our extrapolation base.

MR. STALL:

Thank you very much. It is possible that large storms could change the nature of the gully system such as to set up an entire new system or pattern of gullies. Apparently your data might have included such an event, but you apparently have not evaluated it to that degree.

MR. PIEST:

You are mostly concerned with channel erosion — the additional erosion that would be contributed by new channels — is that right? Generally speaking, on these small watersheds, many channels are well developed. Except in the more arid regions, I would really minimize the total influence of newly developed channels from extreme storms on the sediment yields (that is, for the watersheds I dealt with). On some of the watersheds in western Nebraska that I know of, channel erosion is significant. The channel erosion for the 20-sq. mi. area of Dry Creek near Curtis, Nebr., for example, is very high, but few of the watersheds in the eastern part of the country had much channel erosion as compared with sheet erosion and rill erosion.

MR. STALL:

Thank you. The next question is to Mr. Bullard, who told us about the release of a large sediment load into a trout hatchery pond. "What is the quantitative evidence of the effect of sediment on the trout in this fish hatchery?" This is a subject that many are interested in, and perhaps you can tell us what happened there.

MR. BULLARD:

I don't know about the effect on the trout. The only effect I saw was on the hatchery superintendent, and he was madder than hell! Seriously, though, I very much wish that in connection with some of these studies, which are being made by various agencies on the effects of this land use and that land use on sediment in the streams, that we could get together with the wildlife people in running studies on the effect of sediment on the aquatic habitat and the organisms therein. I know of a couple of such studies.[1] I was talking with one of the registrants here earlier about this. An aquatic biologist by the name of John Peters, who is in the research end of the Montana Fish and Game Department, has done some work that involves the effect of sediment both from range land and from irrigated croplands on a couple of trout streams (I think in the Missouri Basin — some headwater streams). This very interesting piece of work, which I think is just coming to a conclusion, will probably be reported in one of the wild life journals soon. Some similar work is being done in the Water Research Institute at Oregon State University, Corvallis, by Charles Warren who is also an aquatic biologist. I think at the moment Dr. Warren is more concerned with nutrient loading than sediment loading, but the sediment angles are coming in for study also. This study that was mentioned up in Michigan — maybe there was a cooperative study with the wild life people — but I think that would have afforded an excellent opportunity to learn the other side of this story and it is one that is badly needed.

MR. STALL:

Thank you. We have a couple of questions here that we may have to skip, but I would like to ask Emmet Laursen to come up at this time. He doesn't have exactly questions, but he asked for a little time to comment on a different approach to sediment yield. You have about one minute, Emmett.

[1] CORDONE, A. J., and KELLEY, D. W. THE INFLUENCES OF INORGANIC SEDIMENT ON THE AQUATIC LIFE OF STREAMS. Calif. Fish and Game 47. 1961.

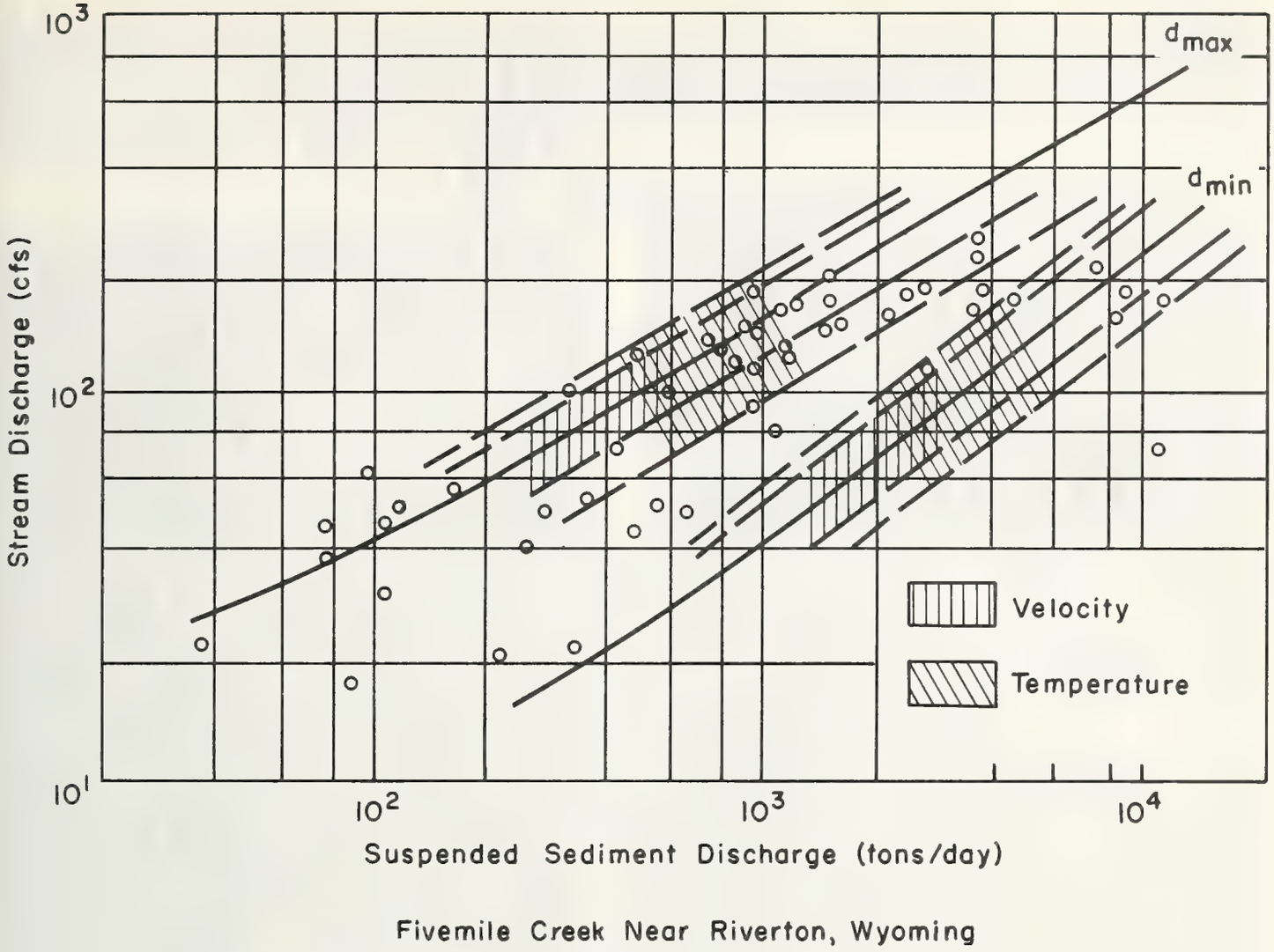


FIGURE 1. — Relation between sediment load and stream discharge at Five Mile Creek near Riverton, Wyo.

MR. LAURSEN:

The accompanying figure is from a Master's study, which attempted to explain the scatter in the relation between sediment load and stream discharge commonly found with natural streams. The example is from Five Mile Creek in Wyoming. (A complete account of the study can be found in the January 1963 issue of the ASCE Journal of the Hydraulics Division in the paper "Sediment-Transporting Characteristics of Streams," by G. A. Zernial and E. M. Laursen.) Data collected by the U.S. Geological Survey on streamflow, bed material, temperature, and sediment load were used. The two heavy curves are the prediction of sediment load if maximum and minimum probable size of bed material and average streamflow and temperature conditions are assumed. Approximately a 10:1 ratio in sediment load can be expected from this change in bed material. The other curves show variable, rather than the sediment yield itself, and come up with a usable relationship. Certainly, the material added should be related to the soil type, the cover, the topography, and the storm. By breaking the total problem of sediment yield up into relatively independent parts, it may be possible to apply more of what is known about rainfall, runoff, and stream behavior and thus, understand sediment yield better.

MR. STALL:

Thank you very much. Gentlemen, I hate to stop but I feel that we should. There are a few of you with unanswered questions. I will refer you to the authors of these papers who are by now pretty well identified and will be here for the next couple of days. We can assure you that all 91 papers of this conference will be published and will be available to you. So, in conclusion of this recap of today's symposium, I would like to comment that I am impressed by the things we do know about sediment movement. We urge that all these things which we have talked about be considered as the building

bricks, which Dr. Vanoni mentioned yesterday, and which can ultimately reveal to us the basic structure of understanding of sediment movement which we all desire. With that, I will turn the meeting back to our chairman.

MR. MURPHY:

Thank you, Mr. Stall, and thanks to all the people who had questions. We were concerned with whether our effort to get questions would be productive, but you set our concern to rest. In fact, we had to run 15 minutes overtime to take care of the questions. I do want to thank the speakers on the program this afternoon for their cooperation in sticking to the schedule, in being brief, and in making very fine presentations. I also feel that the audience deserves commendation, too. You have been very patient in sitting here all day from 8 a.m., including this 15 minutes of overtime, and for maintaining a high level of interest. I believe you are to be congratulated. I haven't seen many people leaving. This was a test to your interest in this subject. I appreciate your attention and your interest and with that I am going to turn this over to the man from the Sedimentation Subcommittee, Mr. Berk.

MR. BERK:

Thank you, Mr. Murphy. I have just one announcement I am going to make. Five of the papers scheduled for distribution are now available — Papers Nos. 8, 29, 37, 95, and 96. They are now available with the other papers. I apologize to Mr. Thatcher of the AEC and to Professor Anderson of Mississippi State College, but we just can't have any more announcements because we are overtime. The Sedimentation Subcommittee declares this session adjourned.

Symposium 2.—Sediment in Streams

INTRODUCTION

During the 15 years that have elapsed since the 1947 Sedimentation Conference, some very significant contributions have been made to the knowledge and understanding of sedimentation in streams. Perhaps the impetus for this progress has resulted largely from the pressure placed on the planners and designers by the rapidly expanding river basin and watershed programs.

The establishment of information on alluvial channels and the development of methodology for handling sediment problems in the field has been attained from investigations, evaluations, and experience in the field and concomitant research on specific elements of sediment transport phenomenon in both laboratory and field. The papers presented in this Symposium, reflecting these advances, provide current information on (1) research on the mechanics of sediment transport in alluvial streams; (2) the relationship between channel roughness and sediment and water conveyance; (3) means of applying theories and equations on total sediment transport in aspects of stable channel design; (4) local scour and channel aggradation and degradation; and (5) engineering practice for handling sedimentation in channel rectification and control.

One of the most notable results of the laboratory investigations has been the confirmation of the observed discontinuity in the stage-discharge relations in alluvial channels. Equally significant in advancing the understanding of sediment transport phenomena has been the development of the Einstein bedload function. The interaction between streambed configuration (bed forms), the hydraulic factors, and the sediment transport has been clearly established from both laboratory and field data. Other significant advances reflected in the Symposium papers include (1) increased awareness of the need to consider the stream as a whole (hydraulic factors, discharge pattern, and total sediment load) in stable channel design; (2) development and application of radioisotope techniques in sediment transport research in the laboratory and in the field; (3) improved procedures for determining total sediment load; (4) advanced designs for handling sediment loads at bifurcations; and (5) criteria or guides for design and installation of compatible control works in alluvial channels.

The papers also indicate that a number of important aspects of sediment transport need further attention through research and evaluation. Some of the unanswered questions for additional research are (1) the prediction of the resistance factors in natural streams by application of laboratory findings; (2) accurate measurement or calculation of total sediment transport for the wide variety of situations encountered in the field; (3) techniques for transposition of data and findings from gaged to ungaged channels; (4) prediction of aggradation and degradation; and (5) extension of limited laboratory studies to encompass a greater range of hydraulic factors, grain size, and flow variability.

SEDIMENT DIVERSIONS THROUGH DISTRIBUTARY CHANNELS NORMAL TO A MAJOR RIVER

[Paper No. 26]

By GEORGE A. PRICE, JR., *engineer, U.S. Army Engineer District, New Orleans*

Synopsis

This paper discusses sediment diversions in three normally oriented distributary channels of the Lower Mississippi River.

Old River is a natural distributary, leaving the Mississippi River at river mile 302. The sediment concentration diverted from the Mississippi River is similar to the concentration in the river for the periods studied. The sediment transport capacity of Old River is somewhat greater than the sediment diverted. This diversionary channel has a history of gradual enlargement.

The Old River Control Structure diversion channel is on the right bank of the Mississippi River at mile 312. The diversion will be regulated by a control structure. Data collected during a trial operation of the control structure

indicated that the diversion channel is in approximate equilibrium. The concentration of sediment diverted was similar to, but slightly less than, the concentration in the river, and the transport capacity was also slightly less. No significant channel changes occurred during the observation period.

The Bonnet Carre Spillway is a gated flood outlet located on the left bank of the Mississippi River at mile 128, and is operated infrequently during major floods. Due to its wide and shallow configuration, the sediment transport is considerably less than the sediment diverted; consequently, sediment deposition occurs.

Introduction

Distributary channels that are oriented perpendicular to a sediment-bearing river frequently present the engineer with silting problems that are much more severe than those that are encountered in the parent stream. Such a distributary may divert a heavier sediment load in proportion to the water discharge than is carried by the river. Unless the diversionary channel is capable of transporting the heavier sediment concentrations, rapid shoaling may result.

This paper will discuss three normally oriented distributary channels of the Lower Mississippi River. In two of the examples the transport capacity of the distributary channel is adequate for the diverted sediment load; in one example the transport capacity is insufficient to avoid heavy sediment deposition.

Problems of this nature are important to the design of artificial diversions, as well as to an understanding of natural diversions of large rivers.

Pertinent Factors

The sand load transported by a stream is a function of the availability of material and the capability of the stream to transport it. Usually the supply of material in the bed is ample in comparison with the capability of the stream to transport it. Thus the bed material (sand) that is transported in suspension and as bedload is a function of the ability to transport the sizes of material found in the bed.

The so-called wash load of silt and clay material carried in suspension is almost entirely a function of supply, since the ability of the stream to transport such material is relatively great. As a general rule the wash load that is transported is equal to the wash load entering the reach.

In the case of a diversion channel, the amount of material entrained in the water that is diverted from the river is also of importance. The diverted sediment concentration may differ from the average concentration in the river if most of the flow is drawn from one part of the cross section. In the case of a bank diversion, the slow-moving water near the bank and near the bed is diverted more readily than faster flowing water. The concentration of sediment near the bed is usually higher than elsewhere in the cross section.

If the entrained load matches the transport capacity of the diversion channel, no material will be removed from or deposited on the bed of the diversion channel. However, if the transport capacity of the diversion channel is significantly different, scour or deposit will occur.

Distributary Channels Normal to the Lower Mississippi River

Old River is a major distributary of the Mississippi River, leaving the Mississippi at river mile 302 and joining the Red River 6 miles to the west to form the Atchafalaya River. Old River at present carries about 25 percent of the flow in the Mississippi River above the junction; the other 75 percent is carried by the Lower Mississippi River past Baton Rouge and New Orleans to the Gulf of Mexico. Enlargement of Old River and Atchafalaya River has been so rapid that it was feared that in one or two decades that route would become the principal outlet of the Mississippi River to the Gulf. Old River is scheduled to be permanently closed in the spring of 1963, when the Old River Diversion Channel and Control Structure are put into operation.

The newly built Old River Control Structure is on the right bank of the Mississippi River at mile 312. Its purpose is to regulate the volume of flow into the artificial diversion channel that parallels Old River and empties into Red River. The structure contains a 73-gate overbank section for floodflows and an 11-gate, low-sill section for low and medium flows. The entire structure is designed for a maximum diversion of 700,000 c.f.s. The low-sill structure was operated during several periods in 1961-62 in order to determine operational characteristics.

The Bonnet Carre Spillway is a gated flood outlet located on the left bank of the Mississippi River at mile 128. Its purpose is to protect the city of New Orleans from flooding, and it has been used three times for that purpose. Its design capacity is 250,000 c.f.s. It is about 2 miles wide by 6 miles long and empties through Lake Pontchartrain into the Gulf of Mexico.

All three of the channels described are normally oriented distributaries of the Lower Mississippi River. Pertinent characteristics such as flow, velocity, depth, sediment concentration, and sediment load have been studied and will be described. The geographic locations are shown in figure 1.

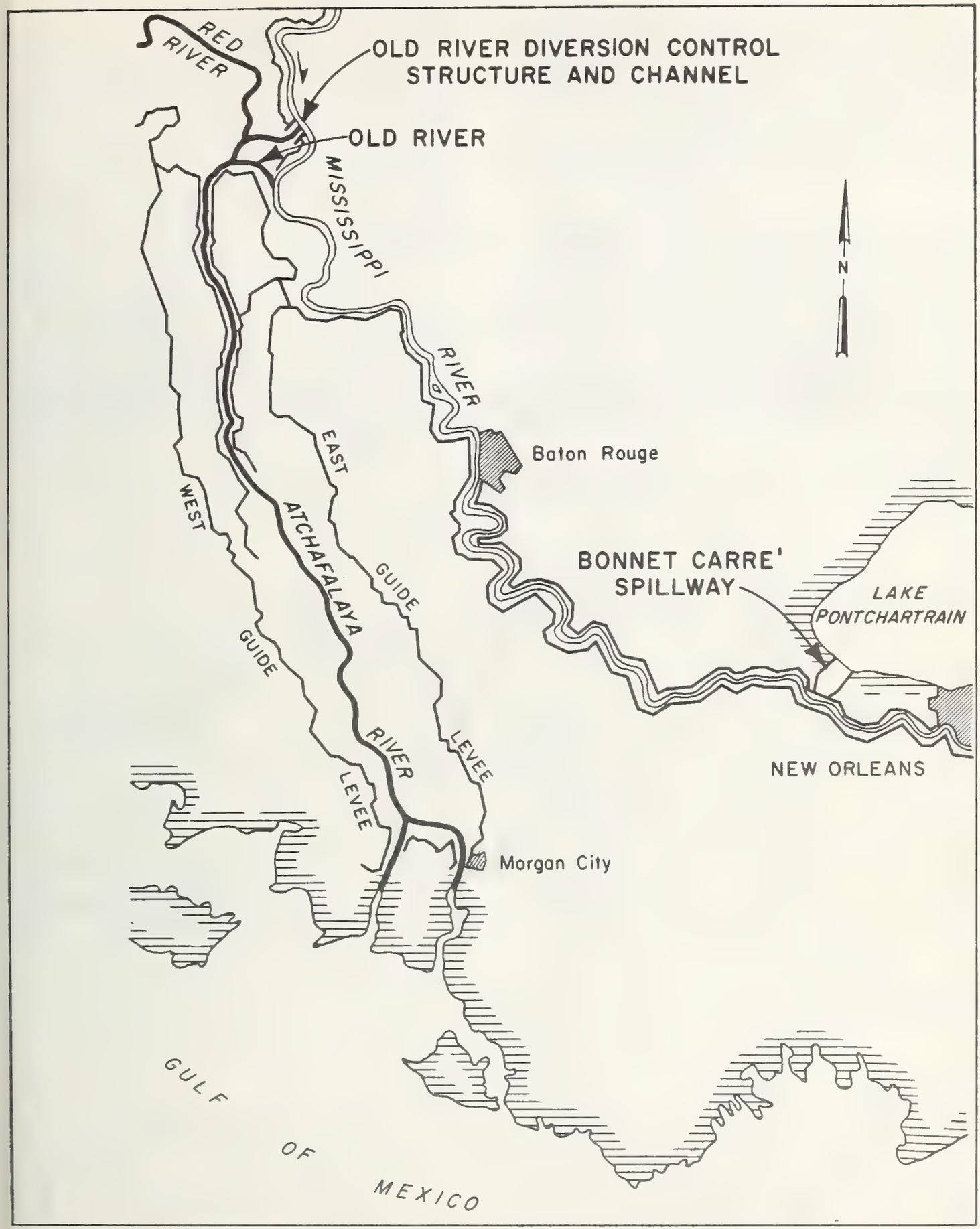


FIGURE 1. — Map showing diversionary channels of lower Mississippi River.

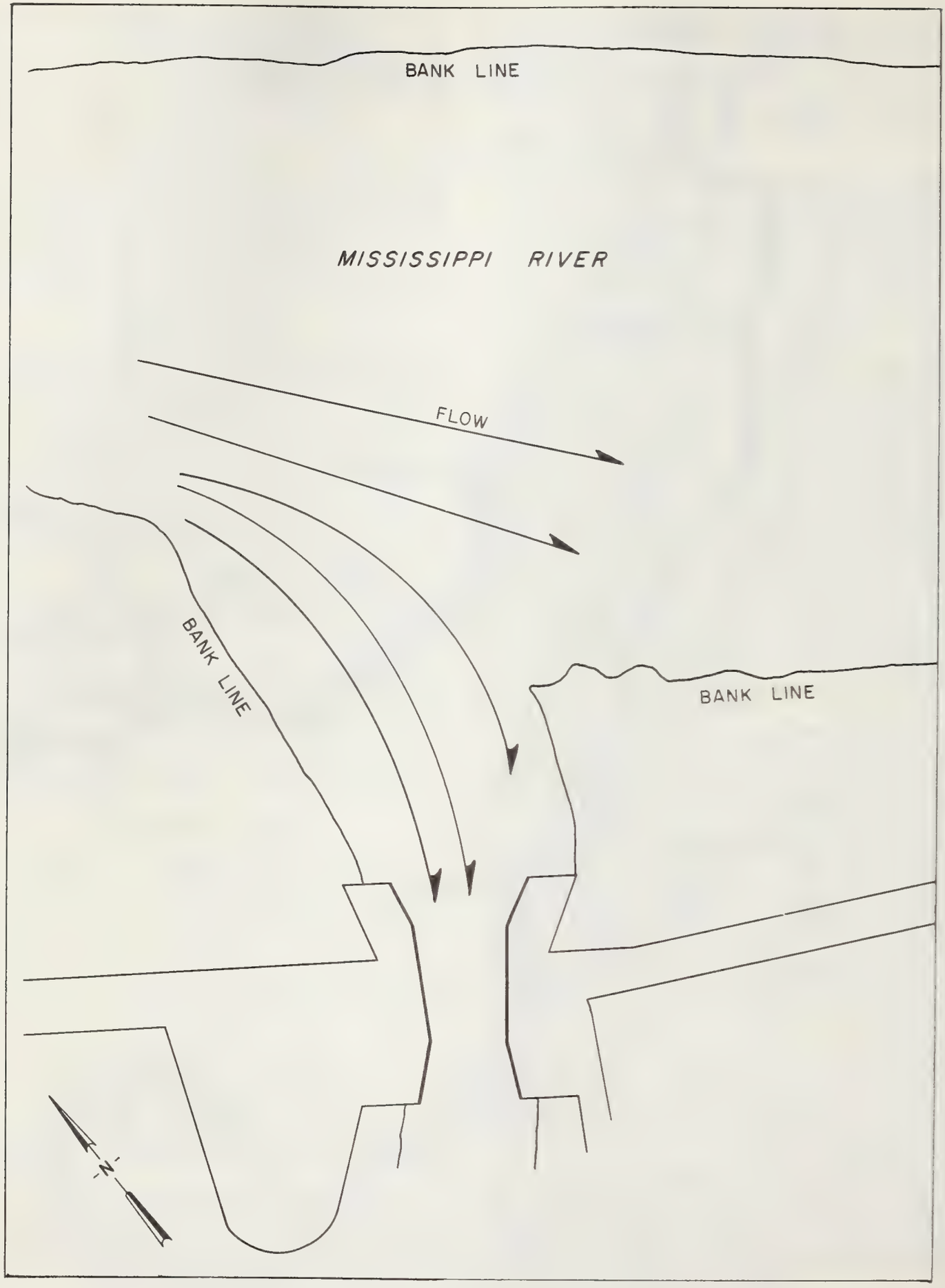


FIGURE 2. — Old River diversion channel and control structure.

Old River Control Structure Diversion Channel

The data selected for study were those obtained in June 1962 during a trial operation of the control structure, and are for flows through only the low-sill diversion structure. A plan view of the diversion is shown in figure 2. At the time of the observations, the Mississippi River was about 50 ft. deep and 3,000 ft. wide, while the diversion channel was about 25 ft. deep and 1,000 ft. wide. The pattern of surface flow into the diversion channel as obtained by float observations is shown in figure 2.

In figure 3 we see the distribution of velocity in the cross section of the Mississippi River above the diversion channel. The elevation of the bottom of the diversion channel is shown for comparison. It may be assumed that flow is diverted from the left one-third of the cross section and from depths extending below the bottom elevation of the diversion channel. The distribution of sediment concentration is also shown in figure 3. The area above the mean concentration line contains lighter-than-average sediment concentrations, whereas the area below the line contains heavier-than-average sediment concentrations. In this example the water diverted appears to come about equally from areas of lighter- and heavier-than-average sediment concentration.

Pertinent observed and computed data are given in table 1. Observed suspended sediment concentrations were obtained with standard field and laboratory procedures and show separately the material suspended as sand (coarser than 0.062 mm.), silt (finer than 0.062 mm.), and total. The volume of water diverted during the period of observation was about 12 percent of the river discharge, and the depth and the velocity in the diversion channel were each

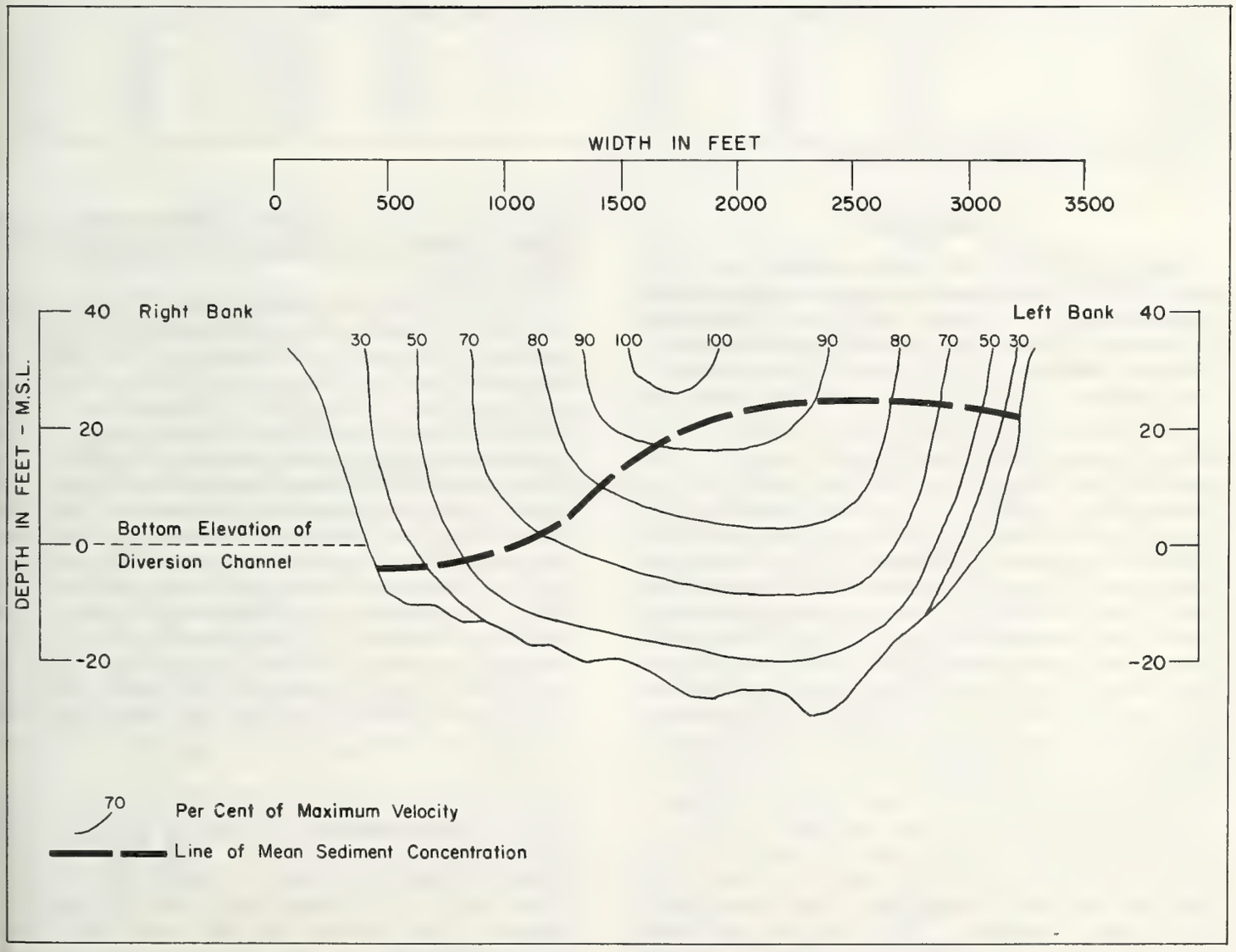


FIGURE 3.— Cross section of Mississippi River above diversion channel.

about one-half the value of that obtained in the river. The sediment loads computed by the simplified Einstein procedure as modified by Colby and Hubbell include suspended load and bedload. The computed concentrations likewise include suspended and bedload material. The computed concentrations were considered to represent the transport capability of the stream.

The data indicate that the concentration of sediment diverted from the river is consistently less, by a small amount, than the concentration in the river. The transport capacity of the diversion channel was also slightly less, due to lesser depths and velocities. Accordingly, the diversion channel appeared to be in approximate equilibrium during the period studied. Surveys confirmed that no overall changes occurred during that period, although there were some localized scour and fill.

TABLE 1.—*Sediment data, vicinity of Old River Diversion Channel, 1962*

MISSISSIPPI RIVER ABOVE DIVERSION, COOCHIE, LA.

Date	Velocity	Depth	Observed concentrations, material in suspension			Computed concentrations, suspended plus bedload [1]		
			Total	Sand	Silt	Total	Sand	Silt
	F.p.s.	*Ft.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
31 May	4.1	51	442	35	407	468	60	408
7 June	4.4	51	389	55	334	423	99	324
14 June	4.8	50	1,148	69	1,079	1,189	117	1,072
21 June	5.0	51	.....	.....	.....	(985)	186	(800)
29 June	4.4	51	688	39	646	730	108	622
DIVERSION CHANNEL IN ENTRANCE TO CONTROL STRUCTURE								
31 May	1.9	20	362	39	323	(385)	60	(325)
7 June	2.2	21	296	28	268	323	58	265
14 June	2.0	23	989	27	962	1,018	64	954
21 June	2.3	30	758	21	737	802	45	757
29 June	2.1	22	.....	.....	.....	(610)	35	(575)

[1] Figures in parentheses are estimated.

Old River

Two periods of data were selected for study, one in 1953 when observations were made at the junction of Old River with the Mississippi River and a period in 1962 with observations at Barbre Landing in Old River, 6 miles west of the junction. (Refer to fig. 1 for the locations.)

The depths and velocities in both streams were similar. Old River diverts about 25 percent of the discharge, and the width is correspondingly narrow. The distribution of velocity in the Mississippi River just above the diversion is shown in figure 4. Flow is diverted from about the left one-third of the cross section. The distribution of sediment concentration is also shown in figure 4. The area above the mean concentration line contains lighter-than-average sediment concentrations, whereas the area below the line contains heavier-than-average concentrations. The water diverted appears to come about equally from areas of lighter and heavier sediment concentrations.

Pertinent observed and computed data are shown in table 2. Data are presented for the two locations in Old River and for Red River Landing on the Mississippi River immediately below the diversion. The observed sediment concentrations are shown for sand (coarser than 0.062 mm.), silt (finer than 0.062 mm.), and total. The computed concentrations are shown for the same classes of material, but include bedloads.

The 1953 observations that were made at the point of diversion show some sediment concentrations higher and some lower in Old River as compared to the Mississippi River, but generally similar. The 1962 observations in Old River at Barbre Landing, 6 miles west of the point of diversion, show generally heavier concentrations in Old River than in the Mississippi River. It was concluded on the basis of the data studied that the sediment diverted at the junction is similar to the concentration in the river, but the sediment transport capacity of Old River is greater than the sediment diverted and additional material is entrained in Old River between the point of diversion and the observation point located 6 miles to the west. The enlargement of Old River has been confirmed by comparative surveys extending over a number of years.

The permanent closure of Old River is scheduled during the spring of 1963. Its function will be replaced by a navigation lock and the controlled diversion structure mentioned earlier. These works were constructed for the express purpose of controlling the flow into and hence the enlargement of the Old River-Atchafalaya River outlet.

TABLE 2.—*Sediment data, vicinity of Old River, 1953 and 1962*

MISSISSIPPI RIVER BELOW OLD RIVER, TED RIVER LANDING, LA.

Date	Velocity	Depth	Observed concentrations, material in suspension			Computed concentrations, suspended plus bedload		
			Total	Sand	Silt	Total	Sand	Silt
	F.p.s.	*Ft.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
23 Jan 1953.....	4.1	27	840	140	700	933	216	717
5 Mar 1953.....	4.6	31	874	385	489	1,006	487	519
28 Mar 1953.....	4.6	36	772	177	595	904	326	578
14 Aug 1962.....	4.0	17	316	16	300	370	69	301
27 Aug 1962.....	3.6	16	264	9	255	369	104	265
10 Sept 1962.....	3.7	17	169	27	142	198	55	143
24 Sept 1962.....	4.1	17	338	28	310	375	62	313
8 Oct 1962.....	4.0	18	266	22	244	311	68	243
OLD RIVER AT JUNCTION								
23 Jan 1953.....	3.4	26	992	200	792	1,098	272	826
5 Mar 1953.....	3.5	34	850	281	569	1,093	495	598
28 Mar 1953.....	3.7	38	875	233	643	988	340	648
OLD RIVER AT BARBRE LANDING, LA.								
16 Aug 1962.....	3.7	18	411	84	327	551	216	335
29 Aug 1962.....	3.2	17	334	35	299	417	107	310
12 Sept 1962.....	3.5	18	189	37	152	254	100	154
26 Sept 1962.....	3.7	22	426	110	316	593	300	293
10 Oct 1962.....	3.3	21	352	95	257	416	149	267

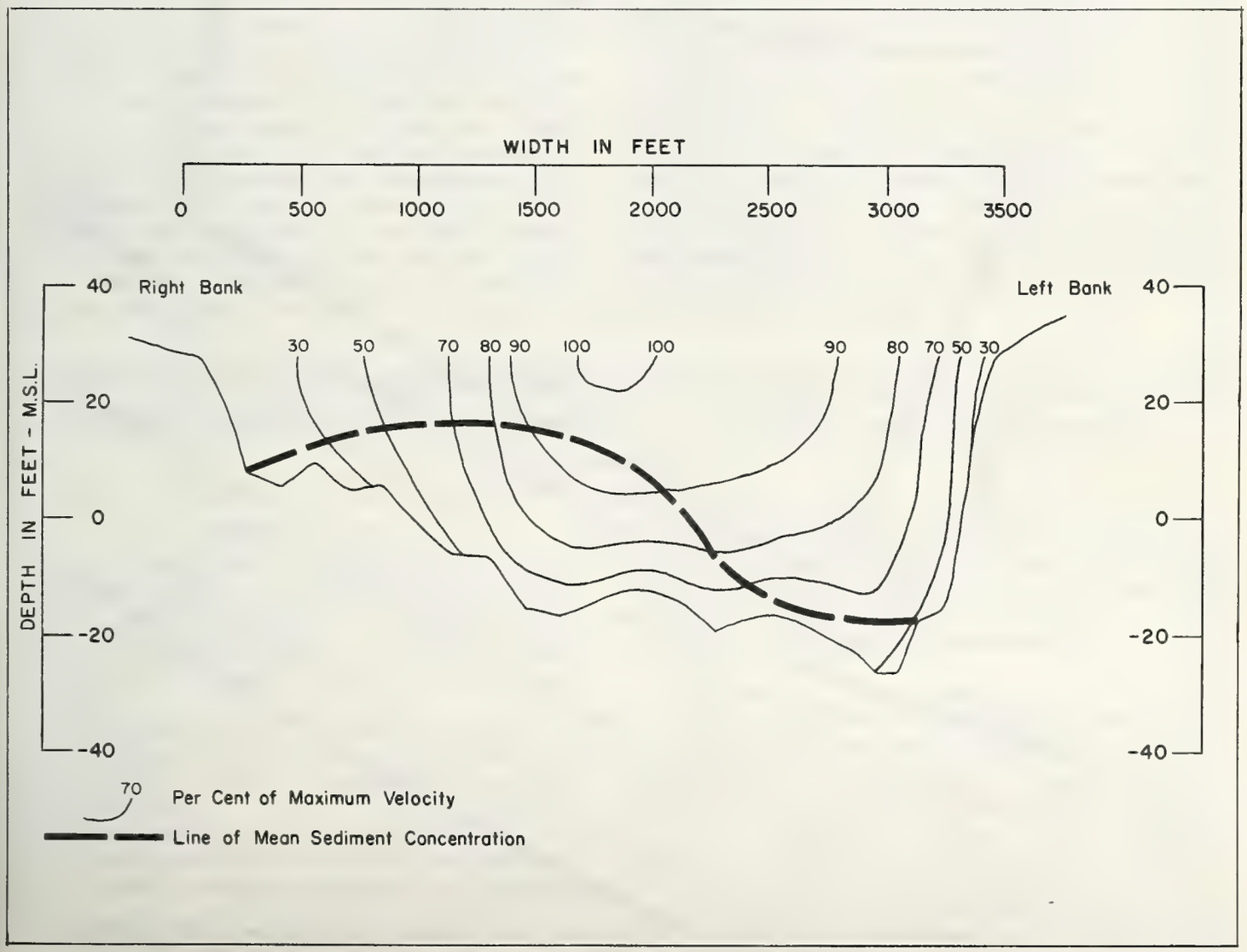


FIGURE 4. — Cross section of Mississippi River above Old River.

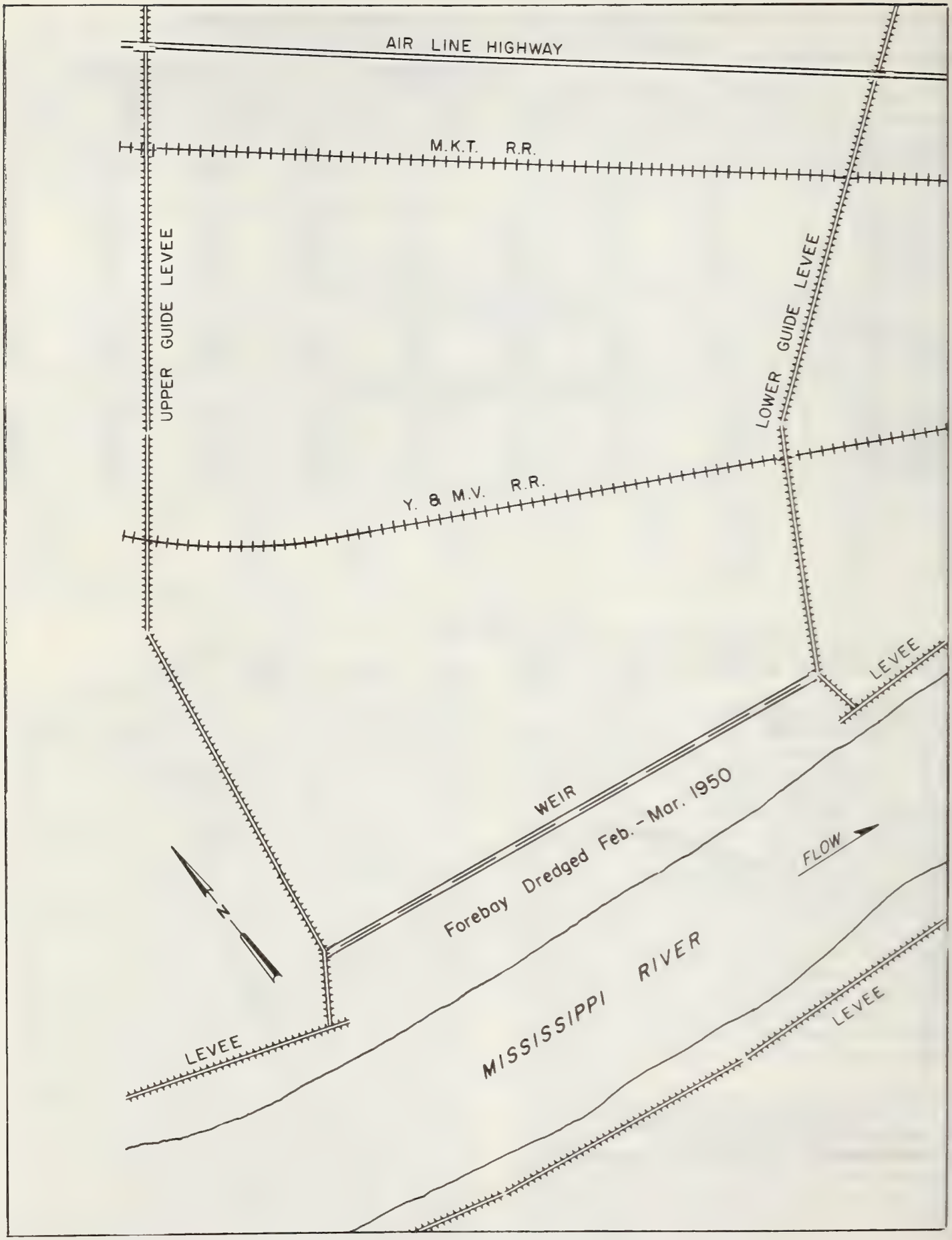


FIGURE 5. — Bonnet Carre Spillway.

Bonnet Carre Spillway

The Bonnet Carre Spillway, described earlier, was operated for a period of 38 days at the height of the Mississippi River flood of 1950. The floodway diversion lowered the flood crest at New Orleans by 2 ft. to assure safe passage of the flood. Over 10 million acre-feet of water was diverted through the floodway. A sketch of Bonnet Carre is shown in figure 5.

The floodway diversion channel was about 2 miles wide and the depth of flow about 10 ft. deep. The Mississippi River just above the diversion was about 2,500 ft. wide and 60 ft. deep. The average velocity in the floodway was 1 to 1.5 f.p.s., and it was about 6 f.p.s. in the river.

The concentration of suspended sediment in the river as measured at Baton Rouge, La., about 100 miles upstream, was 664 p.p.m. average for the 38-day period. Measurements were not made in the Mississippi River directly above the diversion; however, the concentration at the weir at the floodway entrance was 1,068 p.p.m. averaged over the period.

Thus, it appeared that the water diverted carried a heavier suspension of sediment than did the river. However, this conclusion must be qualified by the fact that turbulence at the weir may have carried some bedload into suspension and some material may have been picked up in the forebay. A suction dredge was operating in the forebay, but was discharging its spoil into the river downstream.

Approximately two-thirds of the diverted sediment load was lost in the 6-mile length of the floodway. The concentration at the terminal end was only 400 p.p.m. A total of 5 million cubic yards of deposit remained in the floodway after the operation.

Conclusions

While the data studied are not as complete as could be desired, they indicate:

a. Of the three diversion channels studied, one had a sediment concentration similar to the concentration in the river, one had a somewhat greater concentration, and one a slightly less.

b. In one example the diversion channel was in approximate equilibrium with the diverted sediment load, while one had a greater and one a lesser sediment transport capacity than the diverted load.

c. The distribution of both sediment and velocity in the channel cross section is not uniform, and the pattern of distribution may vary from point to point along the river.

Accordingly, a diversion channel may be in equilibrium with the sediment load carried by the river but may not be in equilibrium with the sediment load diverted from the river. A very thorough study of all pertinent factors is required in the design of a diversion channel that will be in equilibrium, or in determining what changes will occur in a channel that is not in equilibrium. Observations of the distribution of velocity and the distribution of sediment in the river at each location to be considered should be part of the basic study for selecting a suitable diversion site. Such studies will aid materially in obtaining a diverted sediment concentration lighter or heavier than the average in the river, according to the requirements imposed by other factors.

A STUDY OF VARIABLES AFFECTING FLOW CHARACTERISTICS AND SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS

[Paper No. 27]

By D. B. SIMONS[1] and E. V. RICHARDSON, *hydraulic engineers, U.S. Geological Survey, Fort Collins*

Introduction

The U.S. Geological Survey has been studying sediment transport and resistance to flow at Colorado State University, Fort Collins, since 1956. The study has largely been done in laboratory flumes with only limited collection of field data.

The characteristics of the flumes used in the investigations are given in table 1. The principal studies have been completed in the 2-ft. and 8-ft. flumes.

The characteristics (size and gradation) of the bed materials that have been studied, the range of mean velocity, depth, slope, temperature, bed material and fine sediment concentration, and forms of bed roughness investigated and the flume in which each study was made are indicated in table 2.

The investigations conducted in flume 1

[1] Now chief, Civil Engineering Research Section, Colorado State University, Fort Collins.

TABLE 1.—*Characteristics of recirculating adjustable slope flumes used at Fort Collins, Colo.*

Flume No.	Length	Width	Depth	Range of slope	Range of discharge
	Feet	*Feet*	*Feet*	*Percent*	*Cubic feet per second*
1.....	150	8	2	0–1.5	0–22
2.....	60	2	2.5	0–2	0–7.5
3.....	40	⅔	⅔	0–15	0–1.0
4[1].....	200	8	4	0–3	0–100

[1] Under construction.

TABLE 2.—*Flume studies at Fort Collins, Colo.*

Flume No.	Runs	Bed material d_{50}	Gradation coefficient σ	Velocity range	Depth range	Slope-10[2] range	Bed material concentration range	Fine sediment concentration range	Temperature range	Range of bed forms[1]
	Number	*Mm.*		*F.p.s.*	*Ft.*	*Ft./ft.*	*P.p.m.*	*P.p.m.*	*°C.*	
1	41	0.19	1.30	0.74–4.62	0.43–1.09	0.005–0.95	0–47,300	.....	16–19	B–A
1	21	.27	1.54	.73–4.60	.45–1.13	.005–1.022	0–35,800	.....	11–18	B–A
1	38	.28	1.68	.53–4.93	.40–1.07	.005–1.007	0–42,400	.....	10–17	B–A
1	46	.45	1.60	.65–6.18	.25–1.00	.015–1.01	0–15,100	.....	9–20	B–A
1	54	.47	1.54	1.13–5.32	.30–1.33	.084–.96	1.6–17,700	0–42,000	11–22	R–A
1	44	.93	1.54	1.00–5.86	.43–1.11	.0129–1.28	0–10,200	.....	16–21	B–SW
2	30	.32	1.57	.86–5.73	.51–.63	.054–.983	0–29,600	.....	8–33	B–A
2	39	.54	1.52	.89–6.21	.59–.91	.016–1.928	0–50,000	0–102,000	16–23	B–A
2	15	.33	1.27	1.02–5.93	.49–.52	.016–1.14	0–18,400	.....	19.8–20.3	B–A
2	17	.33	2.07	1.06–6.34	.48–.52	.022–.98	0–22,500	.....	18.3–23.4	B–A

[1] B = beginning of motion; R = ripples; SW = standing waves; A = antidunes.

with the 0.19-, 0.27-, 0.28-, 0.45-, and 0.93-mm. bed materials were conceived principally to provide a better understanding of the gross mechanics of flow and sediment transport when a range of sizes of natural alluvial sands is used with a gradation coefficient σ, ranging from 1.5 to 1.6 where

$$\sigma = \frac{1}{2}\left(\frac{d_{50}}{d_{16}} + \frac{d_{84}}{d_{50}}\right)$$

Special studies have been completed to help understand:

(1) The effect of temperature (viscosity).
(2) The effect of large concentrations of fine sediment (bentonitic and kaolin clays).
(3) The effect of the gradation of bed material.
(4) The effect of specific weight of bed material.

For each of the runs for the regular as well as the special studies, the basic data collected included velocity, depth, slope, the size characteristics of the bed material, water temperature, bed roughness (obtained by direct measurement and by the sonic sounder measurements *(13)*), and bed material discharge.

Forms of Bed Roughness

Based upon the general studies, Simons and Richardson (*16*, *18*) suggested the following regimes of flow and forms of bed roughness (fig. 1):

1. Lower Regime
   - Ripples
   - Ripples on dunes
   - Dunes
2. Transition
3. Upper Regime
   - Plane bed
   - Standing waves
   - Antidunes

Resistance to flow is relatively large and bed material discharge is small in the lower regime, and, conversely, resistance to flow is small and bed material discharge large in the upper regime.

The principal reasons for differentiating between ripples and dunes are:

1. Ripples are relatively small, have lengths from 0.25-1.50 ft. and a maximum amplitude of about 0.1 ft., move downstream more or less in the plane of the bed as a result of bedload movement, do not increase in amplitude with increasing depth, have a rather uniform length and amplitude, and do not form under the test conditions when the d_{50} of the bed material is larger than about 0.7 mm.

2. Dunes are similar in appearance to ripples but are much more irregular in form and of considerably greater length and amplitude than ripples. Also, dunes move through a channel at very different elevations with time and are less uniform than ripples. In general, the dimensions of the dunes are quite intimately related to the characteristics of the bed material, particularly fall velocity, but also to slope and depth. Dunes can attain a maximum amplitude approximately equal to the mean depth of flow under ideal conditions. Dunes can occur with any size of bed material so long as the flow brings sufficient tractive forces to bear on the particles to move them at $F_r < 1$. Dunes formed of material with a median diameter of about 2 in., with lengths over 100 ft. and amplitudes of about 3 to 4 ft. have been observed on the lower Rio Grande.

The relation between length of dunes and median fall diameter of bed material for nearly constant depth (fig. 2) shows that the length of dunes increases as size of bed material decreases. Also, the data show that with the smaller sizes of bed material the dunes are less angular.

The resistance to flow caused by the ripples exceeds that for dunes when d_{50} is smaller than about 0.30 mm. under flume conditions. The converse is true for $d_{50} > 0.30$. For field con-

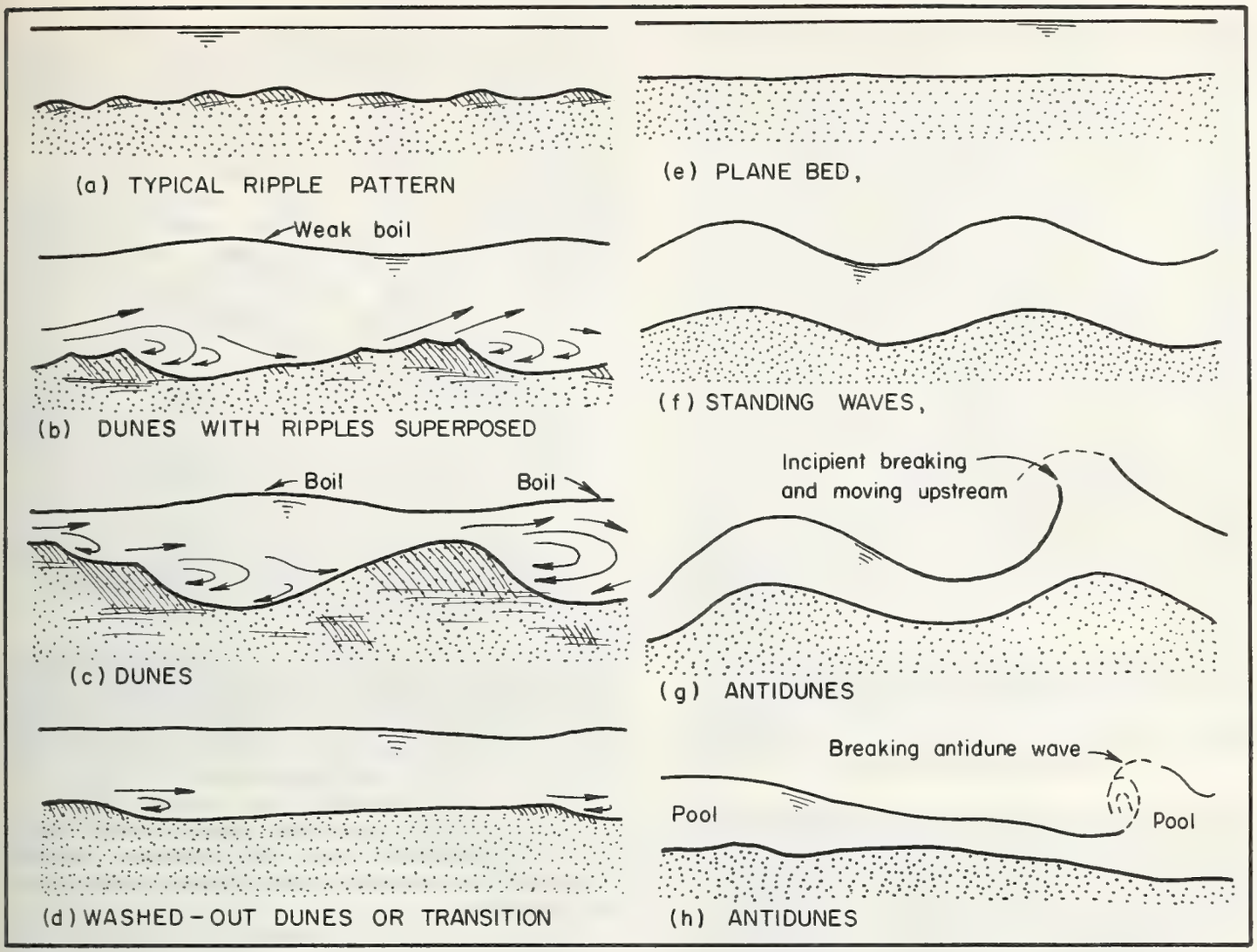


FIGURE 1. — Forms of bed roughness in alluvial channels.

ditions (depths greater than about 2 ft.) the resistance to flow with dunes probably always exceeds that of ripples because of the small relative roughness associated with large depths and ripples. Consequently, resistance to flow caused by ripples and dunes is also a function of their lengths (*21*).

Variables Affecting Bed Roughness

Principal variables that affect the fall velocity and the fall diameter of a given bed material in natural flow, and hence the form of bed roughness, are temperature (viscosity), as reported by Al-Shaikh Ali,[2] and concentration of fine sediment (apparent viscosity), as reported by Haushild, Simons, and Richardson (*10*). Figure 3 indicates the effect of large concentrations of fine sediment (bentonite and kaolin) and temperature on the fall velocity of a river sand with a standard fall diameter of 0.28 mm.

A complete change in form roughness can be accomplished by the introduction of a large concentration of fine sediment (an increase in apparent viscosity of the water-sediment mixture) and, under favorable conditions, by a large reduction in stream temperature (an increase in viscosity of the water). The fact that radical changes in bed form can be caused by large changes in stream temperature or large concentrations of fine sediment, or both, can easily be verified in the laboratory flumes. The same effect has been observed in the field. For example, the Loup River near Dunning, Nebr., has dunes as bed roughness in the summer when the stream fluid is less viscous; whereas, in contrast, the bed is essentially plane during the cold winter months. Similarly, two sets of data collected by Fahnestock[3] in a stable reach of

[2] AL-SHAIKH ALI, KHALID S. INFLUENCE OF TEMPERATURE ON SEDIMENT TRANSPORT AND ROUGHNESS IN ALLUVIAL CHANNELS. Master's thesis. On file, Colorado State University library, Fort Collins. 1961.

[3] FAHNESTOCK, R. K. DATA COLLECTED ON THE RIO GRANDE, 1962. U.S. Geol. Survey, Gen. Hydrol. Written communication.

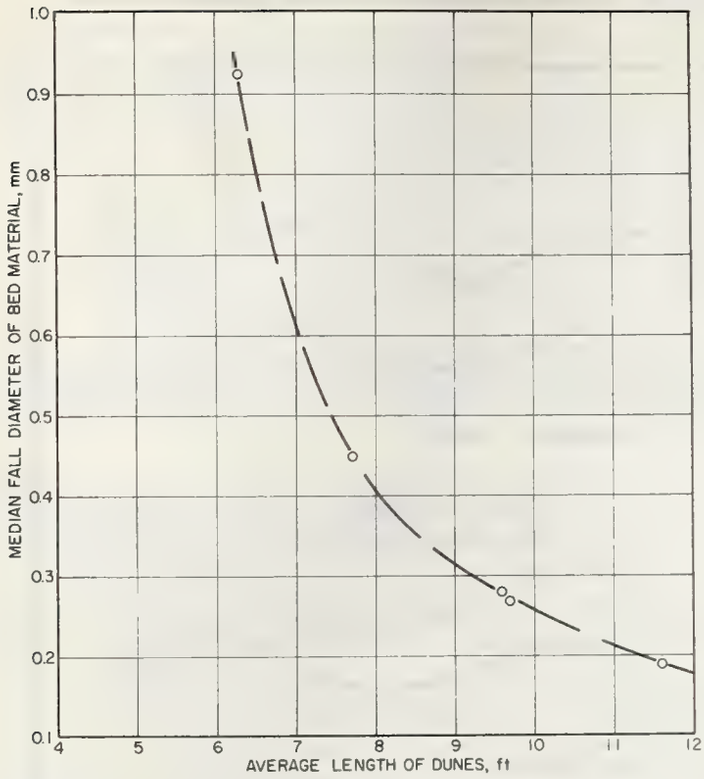


FIGURE 2. — Relation of length of dunes to median fall diameter of bed material.

the Rio Grande at similar discharges show that when the water was cold the bed of the stream was plane, the resistance to flow was small, the depth was relatively small, and the velocity was large; whereas, when the water was warm the bed roughness was dunes, the resistance to flow was large, depth was larger, and the velocity was smaller (table 3).

TABLE 3.—*Rio Grande River data collected on a stable reach for two different stream temperatures at similar discharge*

Temperature	Velocity	Depth	Slope	Bed material	Bed roughness	Manning
F.°	*Ft./sec.*	*Ft.*		d_{50}		*n*
50.....	4.25	2.45	0.00049	0.30	Plane..	0.014
80.....	2.53	3.66	.00053	.30	Dunes..	.034

Changes in the form of bed roughness and, hence, the resistance to flow are reflected in depth and stage-discharge curves. Figure 4 (*19*) shows a typical break in a depth-discharge relation caused by a change in bed form from dunes to plane bed. The Manning *n* for the dune bed was approximately 0.03; whereas, with the plane bed condition the Manning *n* was 0.012.

[4] DARANANDANA, NIWAT. A PRELIMINARY STUDY OF THE EFFECT OF GRADATION OF BED MATERIAL ON FLOW PHENOMENA IN ALLUVIAL CHANNELS. Doctor of Philosophy dissertation. On file, library of Colorado State University, Fort Collins. 1962.

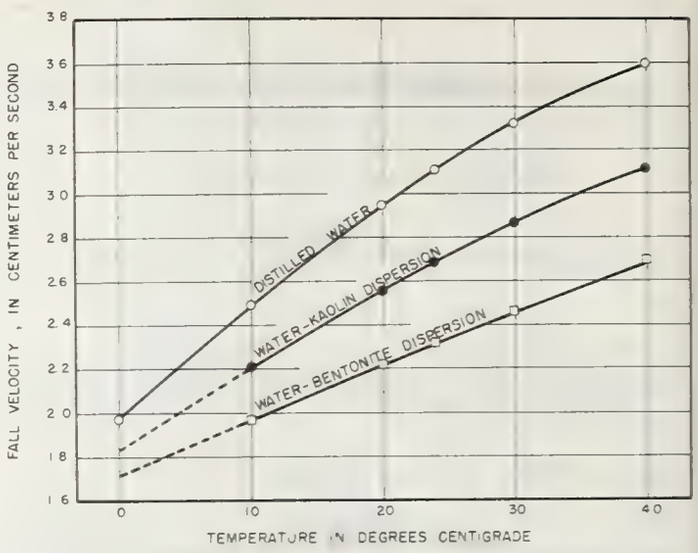


FIGURE 3. — Variation in fall velocity of median particle size of Elkhorn River, Nebr., sand with changes in temperature in distilled water and with 5 percent by weight dispersions of kaolin and of bentonite.

As a further verification of the significance of fall velocity, experiments were conducted in which an expanded clay (idealite) with a specific gravity of about 1.7, a median sieve diameter of 0.7 mm., and a fall diameter of 0.35 mm. was used as a bed material. For a given flow condition this lightweight bed material was shaped into essentially the same bed form as was observed with the use of the 0.33 mm. sand as the bed material.

To determine, in a preliminary way, the role of gradation of bed material with respect to resistance to flow and sediment transport in alluvial channels, two sets of runs were completed by Daranandana, 1962,[4] in flume 2, using two different bed materials and holding depth and temperature constant. The two bed materials had the same median fall diameter, but one was

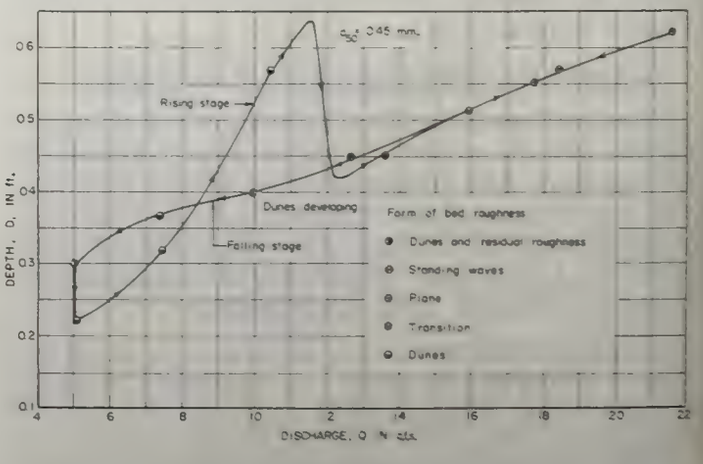


FIGURE 4. — Typical depth-discharge relation involving lower and upper regime flow.

quite uniform ($\sigma = 1.27$) and the other was graded ($\sigma = 2.07$). The effects of gradation on form of bed roughness are qualitatively illustrated in figure 5.

To illustrate further the effect of gradation of bed material on resistance to flow in alluvial channels, figures 6 and 7 (after Daranandana (1962)) relate f_b to V and f_b to S, respectively, for the runs made with the uniform and the graded bed material, $d_{50} = 0.33$ mm. These relations follow logically from figure 5. The large scatter in these relations, when the bed form is ripples and dunes and the bed material is graded, indicates that continual sorting and remixing takes place with time. This suggests that the representative fall velocity and gradation of the bed material also varies with time, which adds to the complexity of the mechanics of flow and transport in alluvial channels.

Prediction of Form of Bed Roughness

A completely satisfactory method of predicting form of bed roughness has apparently never been developed. Various methods of predicting form roughness [Albertson, Simons and Richardson (*1*); Simons and Richardson (*16*); and Garde [5]] have been proposed; however, none of the methods are applicable for both laboratory and field conditions. A relatively simple relation (fig. 8) that relates stream power, median fall velocity of bed material, and form roughness gives an indication of the form of bed roughness one can anticipate if the depth, slope, velocity, and fall diameter of bed material are given. Flume data were utilized to establish the boundaries separating (1) plane bed and ripples, (2) ripples and dunes for all sizes of bed material, and (3) dunes and transition for the 0.93-mm. bed material. The lines dividing (1) dunes and transition and (2) transition and upper regime are based upon the following field data: (1) Elkhorn River (Beckman and Furness (*2*)); (2) Rio Grande 20 miles above El Paso (see table 3); (3) Middle Loup River at Dunning (Hubbell and Matejka (*11*)); (4) Rio Grande at Cochiti; (5) Rio Grande near Bernalillo; (6) Rio Grande—Angostura heading (Culbertson and Dawdy (*7*)); (7) the canal

[5] GARDE, R. J. TOTAL SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1959.

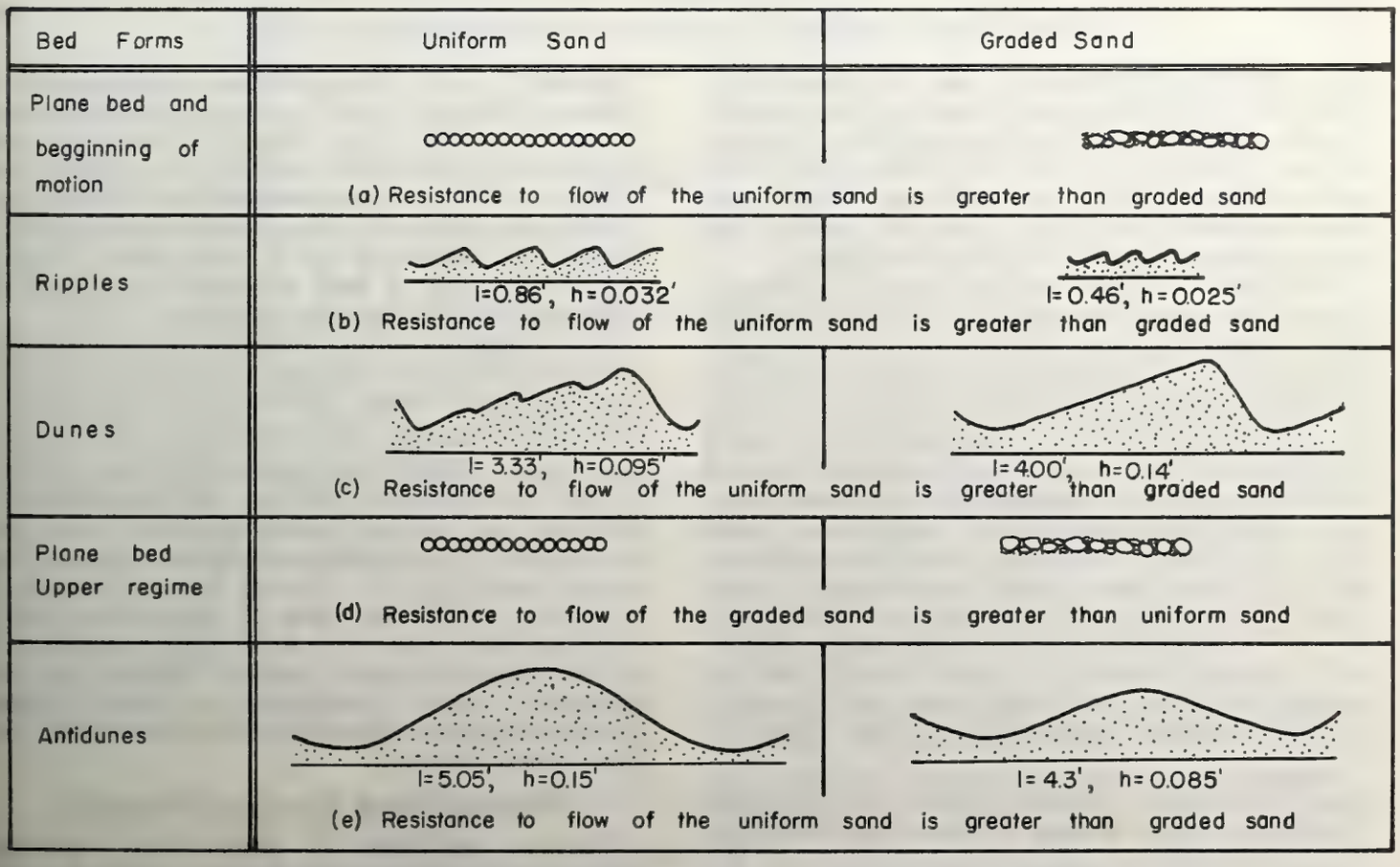

Bed Forms
Uniform Sand
Graded Sand
Plane bed and begginning of motion
(a) Resistance to flow of the uniform sand is greater than graded sand
Ripples
l=0.86', h=0.032'
l=0.46', h=0.025'
(b) Resistance to flow of the uniform sand is greater than graded sand
Dunes
l=3.33', h=0.095'
l=4.00', h=0.14'
(c) Resistance to flow of the uniform sand is greater than graded sand
Plane bed Upper regime
(d) Resistance to flow of the graded sand is greater than uniform sand
Antidunes
l=5.05', h=0.15'
l=4.3', h=0.085'
(e) Resistance to flow of the uniform sand is greater than graded sand

FIGURE 5. — Comparison of resistance to flow for various bed forms for both sands.

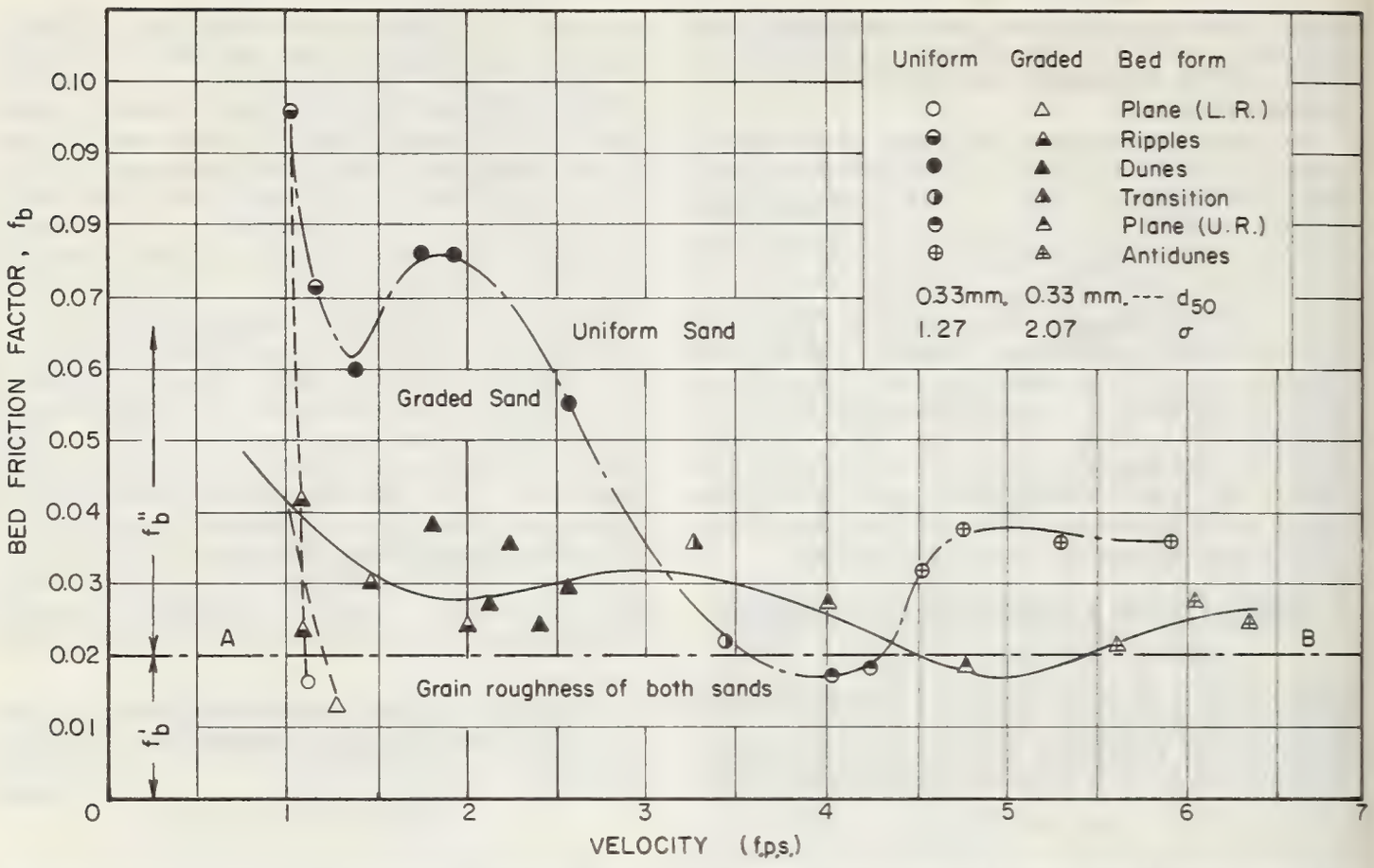


FIGURE 6. — Comparison of total bed friction factor, friction factor pertaining to grains, and friction factor pertaining to form roughness for uniform and graded bed material sands.

data reported by Simons;[6] and (8) canal data collected in Pakistan by Tipton and Kalmbach,[7] Denver, and Harza Engineering Co., Chicago, during 1961-62. If only the flume data were used, the dividing line between dunes and transition would occur at about 10 percent less stream power than the field data indicates.

In figure 8, note that the range of stream power for which dunes occur becomes smaller with decreasing fall diameter of bed material. Hence, when the median fall diameter of the bed material is relatively small, stable channels must be designed with care. For example, if large resistance to flow is anticipated, stable channel design concepts indicate a relatively small average velocity, large depth, and steep slope. However, the channel may not function as anticipated. If the stream power is sufficiently large, the bed form will not be dunes but will be plane, which will result in a small resistance to flow, a high average velocity, a small depth, and an unstable channel with high transport capacity. A good example is the Marala-Ravi Canal in Pakistan.[7] This canal has bed material with a median diameter of 0.29 mm. and was designed to carry 15,500 c.f.s. When it was put into operation its bed depth was 9.1 ft., its slope was 1/3,460, with an average velocity of 4.51 ft./sec. The stream power for these conditions was 0.71, which was sufficient for a plane bed, channel instability, and a bed material discharge of 132 tons per day per foot of width.

The measurement, as well as the prediction of average velocity in alluvial channels, is a complex problem. When the velocity data are reviewed it is immediately noted that error can be introduced into the measurement by any change in bed configuration during the measurement. Also, as the water-sediment mixture flows over the crests of the ripples and dunes, a separation occurs (see fig. 1). Taking measurements of velocities within the zone of separation shows that upstream velocities exist that are one-half to two-thirds average stream velocity and the boundary shear is sufficient to form ripples that are oriented opposite to the direction of the flow in the channel.

Prediction of Average Velocity

Various methods of determining average velocity and resistance factors for alluvial channels have been suggested. The Manning and

[6] SIMONS, D. B. THEORY AND DESIGN OF STABLE CHANNELS IN ALLUVIAL MATERIALS. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1957.

[7] Data collected by Tipton and Kalmbach, Inc., Engineers, Denver, Colo., written communication. 1961–62.

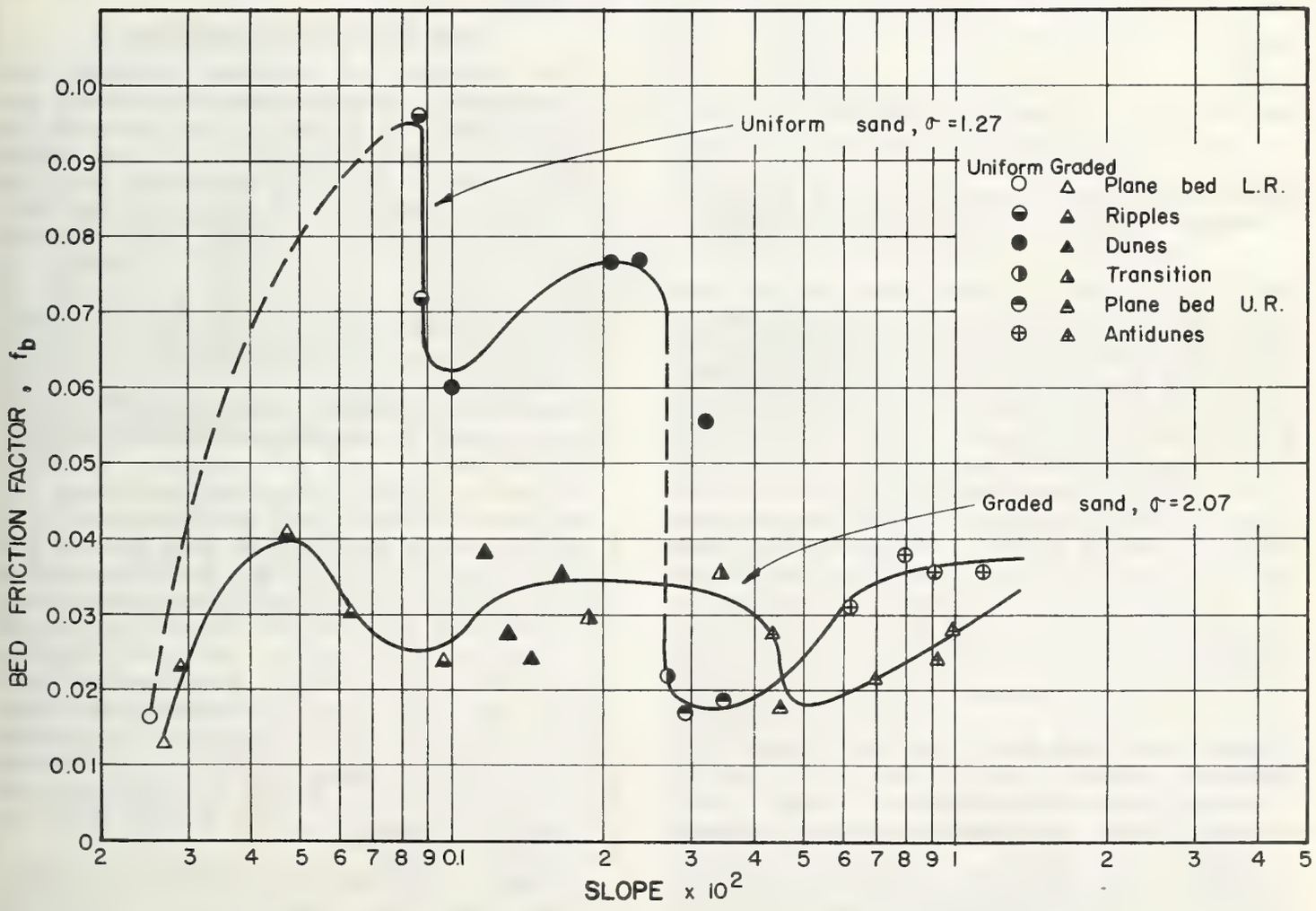
Chezy equations developed for channels with rigid boundaries, the regime equations (*12*) quite commonly referred to when attempting to design stable alluvial channels, and Einstein's and Barbarosa's treatment (*9*) of alluvial river channel roughness have all been used to estimate channel resistance and average velocity. The latter two methods are probably the most successful of those cited.

The flume data referred to in table 2 follow Einstein's river curve, which relates V/V_*'' to ψ', with reasonable success within the range of ψ' values for which ripples and dunes occur. However, for both large and small values of ψ', the data departs somewhat systematically and radically from the proposed curve. Hence, some modification is required when it is applied to laboratory data (fig. 9).

The occurrence of the separation zones downstream of the ripples and dunes suggests that the cross sectional area through which the flow is assumed to occur should be reduced to eliminate the separate zones. This reduces the actual depth D to an effective depth D' and increases the average velocity V to an effective velocity V'. Using these concepts of depth and velocity, Simons and Richardson (*17*) devised a useful method of predicting the average velocity in alluvial channels. The method involved establishing a relation between V and V_* for the plane bed runs for each median fall diameter and neglecting the difference in resistance to flow and relative roughness with and without sediment movement for the plane bed. The resultant relation (fig. 10a), which is essentially a straight line that passes through the origin, implies that for plane bed

$$V \alpha V_* \text{ and } V = \frac{C}{\sqrt{g}} \sqrt{g\,DS}$$

With forms of bed roughness other than plane bed, the points relating V and V_* fall to the right of the relation for plane bed, indicating a greater resistance to flow, the magnitude of which is proportional to the displacement to the right of the line. Assuming that this dis-



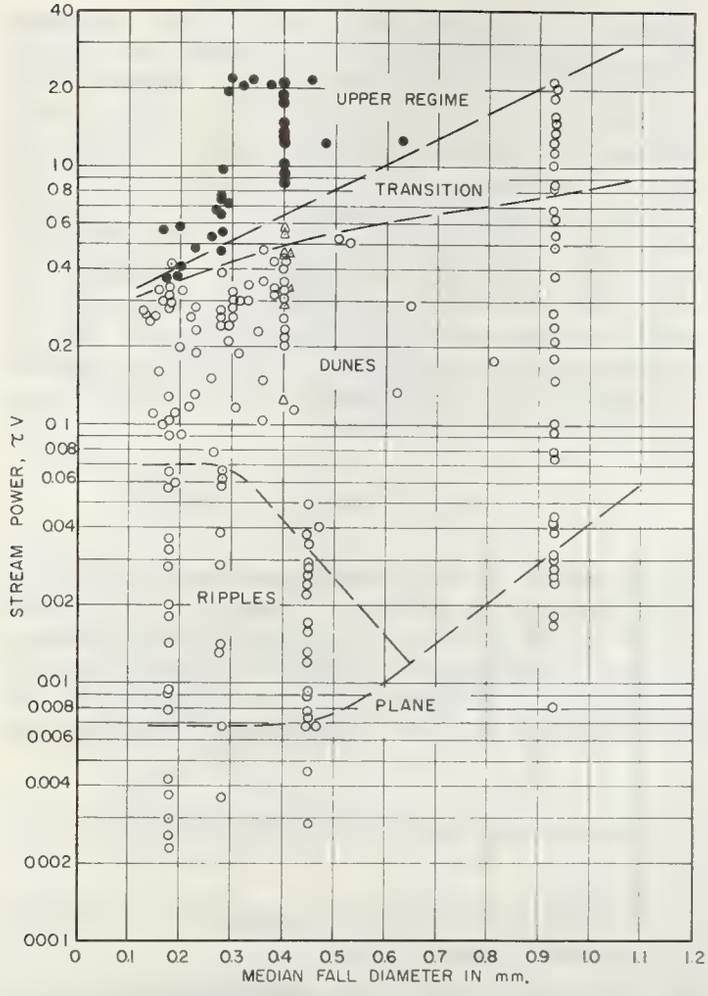


FIGURE 8. — Relation of stream power and median fall diameter of bed material to form of bed roughness.

placement is related to the volume of the flow within the separation zones downstream of the ripples or dunes, a correction to depth and velocity can be made based upon the assumed magnitude of the separation zones which yields an effective depth D' and velocity V'. Replotting the relation between V and V_* for each run with similar bed roughness and using V' in place of V and $\sqrt{g\,D'S}$ in place of $\sqrt{g\,DS}$ puts the new points essentially on the line representing plane bed conditions. This indicates that if the depth correction $\triangle D$ can be determined, making $D' = D - \Delta D$ and $V' = V\ D/D'$, a useful method of predicting average velocity results, provided the form of bed roughness and median fall diameter of the bed material are known. The complexity of predicting form roughness has already been discussed. Also, the usefulness of this method is limited when dealing with a situation where a small change in stream power will cause a change in bed roughness and a large change in average velocity.

The method applied to stable irrigation canals with dunes having median fall diameters of about 0.28 mm. is illustrated in figure 10a. The straight line passing through the origin is based upon plane bed flume runs of bed materials with median fall diameters of 0.27 and 0.28 mm. Figure 10b relates the depth D to the correction ΔD, which is assumed to be related to the volume of the separation zones relative to the total flow and slope of energy gradient. The method of estimating average velocity, using figure 10, is illustrated in table 4. Also in table 4 the estimated average velocity is given for comparison with the measured average velocity. For most of the canals, agreement is within 10 percent and illustrates the effectiveness of the method.

Certainly the source of considerable error in determining velocity and sediment transport is in evaluating the magnitude and effect of each significant variable — only a few have been discussed here. Future fundamental studies of the variables, to define more precisely their effects on flow in alluvial channels, are essential.

Bed Material Discharge

The mechanics of sediment transport are very complex — many empirical and semi-empirical methods of estimating bed material discharge have been proposed. Of these many methods, two of the most commonly used are the Einstein method (*8*) and the modified Einstein method (*6*). Although the modified method is more accurate, its usefulness is limited because computations are based on actual suspended sediment and velocity data; hence, it is not particularly useful for design problems.

The relation between measured bed material discharge and the bed material discharge computed by the Einstein (*8*) procedure is given in figure 11. It was found that agreement between the computed and measured sediment discharge varied appreciably with size of bed material and the form of bed roughness. Best results occurred in the transition zone connecting lower and upper regime flow. The largest variation occurred within the lower regime, particularly near beginning of motion and with ripples where bed material discharge was relatively very small. Success hinges on beginning with a known average velocity for each run. For design purposes, where it is necessary to estimate the average velocity, results would be less impressive.

A study of the Einstein method by Bishop,[8] in which the flume data cited in table 2 was used, resulted in a rather simple method of

[8] BISHOP, A. A. SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS, A CRITICAL EXAMINATION OF EINSTEIN'S THEORY. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1961.

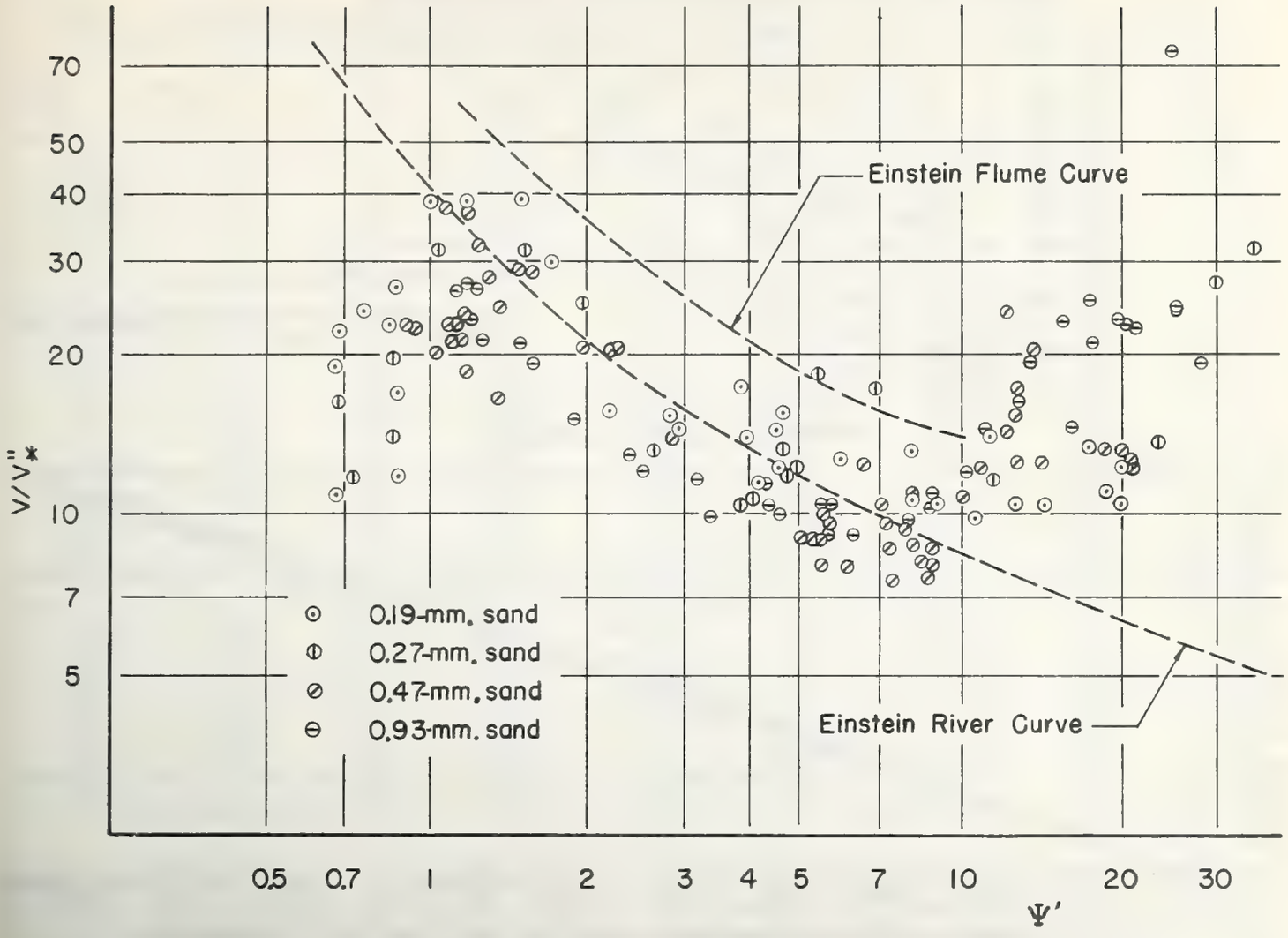


FIGURE 9. — Flume resistance data for various sands in relation to Einstein's bar resistance diagram.

estimating bed material discharge. It involves:

1. The use of the first nine steps as described by Einstein (*8*) to determine ψ', except that R' was determined by a more direct graphical method developed by Vanoni and Brooks (*20*) where

$$\psi' = \frac{\rho_s - \rho_f}{\rho_f} \frac{d_{35}}{R' S}$$

2. Values of ϕ_T in relation to ψ' can be determined from figure 12, from which the total bed material discharge q_T can be estimated from the relation

$$q_T = \phi_T \rho_s g d_{50}^{3/2} \left(\frac{\rho_s}{\rho_f} - 1 \right) 43.2 w$$

The $\phi_T - \psi'$ curves (fig. 12) are very similar to Einstein's $\phi_* - \psi_*$ curves, except that the transport intensity factor

$$\phi_T = q_s \Big/ \rho_s g^{3/2} d_{50}^{3/2} \left(\frac{\rho_s}{\rho_f} - 1 \right)^{1/2}$$

is calculated from the total bed material load, if i_B and i_b are assumed equal to unity. The lower leg of each $\phi_T - \psi'$ curve has been fit by the probability function defined by Einstein. However, A_* and B_* were found to vary systematically with median fall diameter of bed material as shown in figure 13. The upper legs of each of the $\phi_T - \psi'$ curves do not follow the probability relation and have been extended to fit the data empirically. The bed material discharge computed accordingly is compared with the measured bed material discharge in figure 14. The relation is consistently improved throughout the entire range of flow conditions investigated.

To illustrate the possible utility of Bishop's method[9] for estimating bed material discharge in canals and rivers, the total bed material discharge was computed for the (1) canals investigated by Simons,[10] (2) the Niobrara River, (3) the Loup River, (4) the Colorado River, (5) the Solomon River, and (6) the Rio Grande. The computed values are related to a compar-

[9] See footnote 8.

[10] See footnote 6.

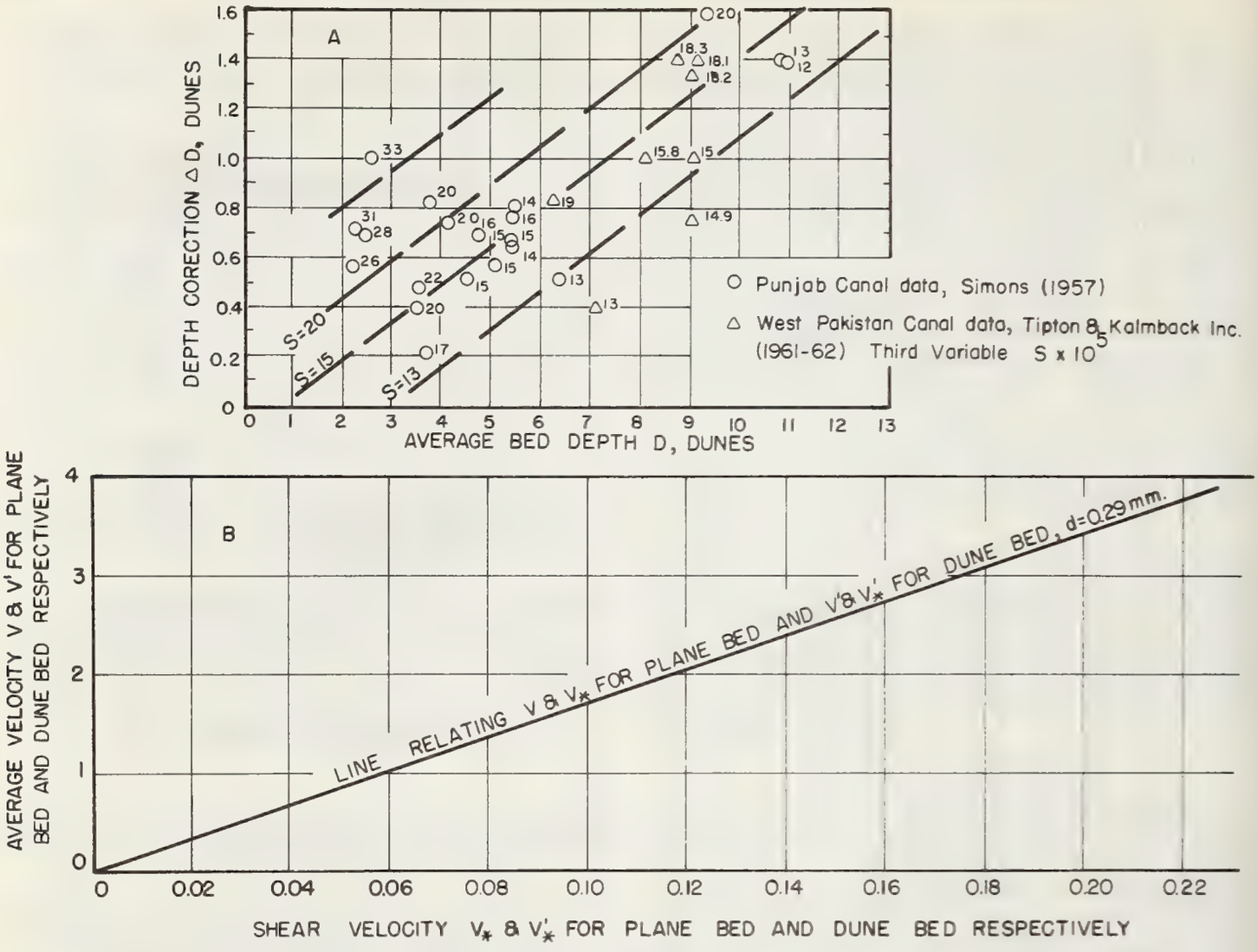

A
DEPTH CORECTION Δ D, DUNES
AVERAGE BED DEPTH D, DUNES
S=20
S=15
S=13
○ Punjab Canal data, Simons (1957)
△ West Pakistan Canal data, Tipton & Kalmback Inc. (1961-62) Third Variable S x 10^5
B
AVERAGE VELOCITY V & V' FOR PLANE BED AND DUNE BED RESPECTIVELY
LINE RELATING V & V* FOR PLANE BED AND V' & V'* FOR DUNE BED, d=0.29 mm.
SHEAR VELOCITY V* & V'* FOR PLANE BED AND DUNE BED RESPECTIVELY

FIGURE 10. — Relation of V and V' to V_* and V'_*, respectively, for bed material with a median fall diameter of about 0.29 mm. for plane bed and dunes.

able total bed material discharge obtained by (1) direct measurement of the total bed material load in the canals, (2) direct measurement at a contracted section for the Niobrara River, (3) by direct measurement at the turbulence flume (*3*) on the Loup River and by the modified Einstein method for the Colorado River, the Solomon River and the Rio Grande (fig. 15). The agreement is best for shallow streams but offers good possibilities for all streams. However, as is the case for the flume data the success achieved, when the method to field conditions is applied, is dependent on the accuracy with which the stream velocity can be estimated or measured.

Future Studies

A series of experiments are being considered, oriented toward:

1. A study of the mechanics of the formation of bed roughness in alluvial channels and a statistical analysis of their occurrence.
2. Determination of the magnitude of lift and drag on sediment particles.
3. Determination of fall velocity of gravel particles.
4. Measurement and evaluation of the role of turbulence on resistance to flow and sediment transport in alluvial channels.

The study of bed roughness will be initiated in flume 3 with a lightweight bed material (Vestyron) used with a specific gravity of 1.04-1.06. When this facility is used, dunelike roughness elements can be generated by the flow at very small Reynolds numbers. Actually, the individual motion of a single particle can be followed and studied over an appreciable distance.

The magnitude of the lift and drag forces on particles is being studied, using individual beads mounted on a fine wire. The force on the bead is measured with calibrated semi-conductor

TABLE 4.—*Comparison of measured average velocity to computed average velocity, using figure 10 for stable canals with dune bed and median diameter of bed material of approximately 0.29 mm.*

Depth D	Slope $S\times10^3$	ΔD	D'	$\sqrt{gD'S}$	V'	Computed V	Measured V	Percent error
PUNJAB DATA, SIMONS (see footnote 6)								
2.6	0.33	0.9	1.7	0.135	2.44	1.59	1.4	+13.6
3.45	.22	.75	2.70	.138	2.40	1.88	2.16	−12.9
2.24	.31	.76	1.48	.122	2.10	1.39	1.53	− 9.15
2.20	.26	.50	1.70	.119	2.05	1.58	1.50	+ 5.3
3.74	.17	.54	3.20	.132	2.30	1.97	2.28	−13.6
5.10	.15	.66	4.44	.146	2.50	2.17	2.27	− 4.4
5.42	.15	.70	4.72	.151	2.60	2.26	2.30	− 1.8
5.43	.14	.52	4.91	.154	2.65	2.40	2.25	+ 6.7
4.97	.15	.64	4.33	.145	2.50	2.18	2.14	+ 1.9
4.53	.15	.57	3.96	.138	2.40	2.10	2.14	− 1.9
6.36	.13	.53	5.83	.156	2.70	2.47	2.49	+ .8
5.44	.16	.75	4.69	.155	2.70	2.33	2.32	+ .4
2.46	.28	.70	1.76	.126	2.20	1.57	1.56	+ .7
3.50	.20	.67	2.93	.137	2.40	2.01	2.16	− 7.0
3.78	.20	.71	3.07	.140	2.40	1.95	1.88	+ 3.7
5.47	.14	.54	4.93	.149	2.60	2.34	2.15	+ 8.8
4.16	.20	.77	3.39	.148	2.55	2.08	2.12	− 1.9
4.73	.16	.65	4.08	.145	2.50	2.16	2.12	+ 1.9
10.80	.13	1.25	9.55	.20	3.45	3.08	2.84	+ 8.5
10.90	.12	1.20	9.70	.194	3.35	2.99	2.87	+ 4.2
WEST PAKISTAN DATA, TIPTON AND KALMBACH, INC. (see footnote 7)								
7.0	0.132	0.63	6.37	0.169	2.92	2.66	2.74	− 2.9
10.0	.149	1.40	8.60	.203	3.51	3.02	3.36	−10.1
9.0	.15	1.27	7.73	.193	3.34	2.87	3.02	− 5.0
8.1	.158	1.15	6.95	.188	3.25	2.79	2.86	− 2.4
8.7	.183	1.39	7.31	.208	3.60	3.03	2.97	+ 2.01
6.0	.193	1.04	4.96	.175	3.03	2.51	2.67	− 6.0
9.1	.181	1.46	7.64	.211	3.65	3.06	3.10	− 1.3
9.0	.182	1.45	7.55	.210	3.64	3.06	3.10	− 1.3
9.3	.20	1.57	7.73	.223	3.85	3.20	3.15	+ 1.6

strain gages. This method of measuring drag on a wire was originated by Sharp (*15*) to determine velocity in pipes and open channels.

The measurement of turbulence in water has been a difficult task because of many factors — principally inadequate turbulence measuring equipment. However, preliminary work done in flume 3, using the electrokinetic probe reported by Chuang and Cermak (*5*), indicates the feasibility of measuring turbulence in both pipes and open channels. The probe by Chuang (*4*) has also been used in flume 3 to measure the frequency of shedding of vortices downstream of cylinders. The results thus obtained are in excellent agreement with those obtained by Roshko (*14*) in a wind tunnel with a hot wire anemometer.

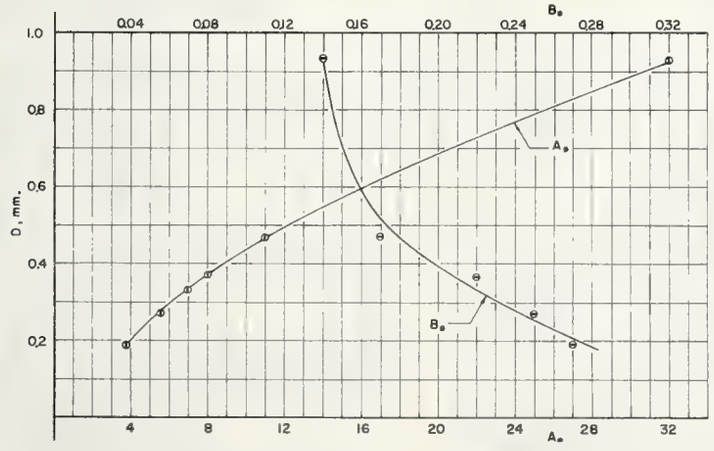


FIGURE 11. — Measured sediment discharge compared to sediment discharge computed by Einstein's function.

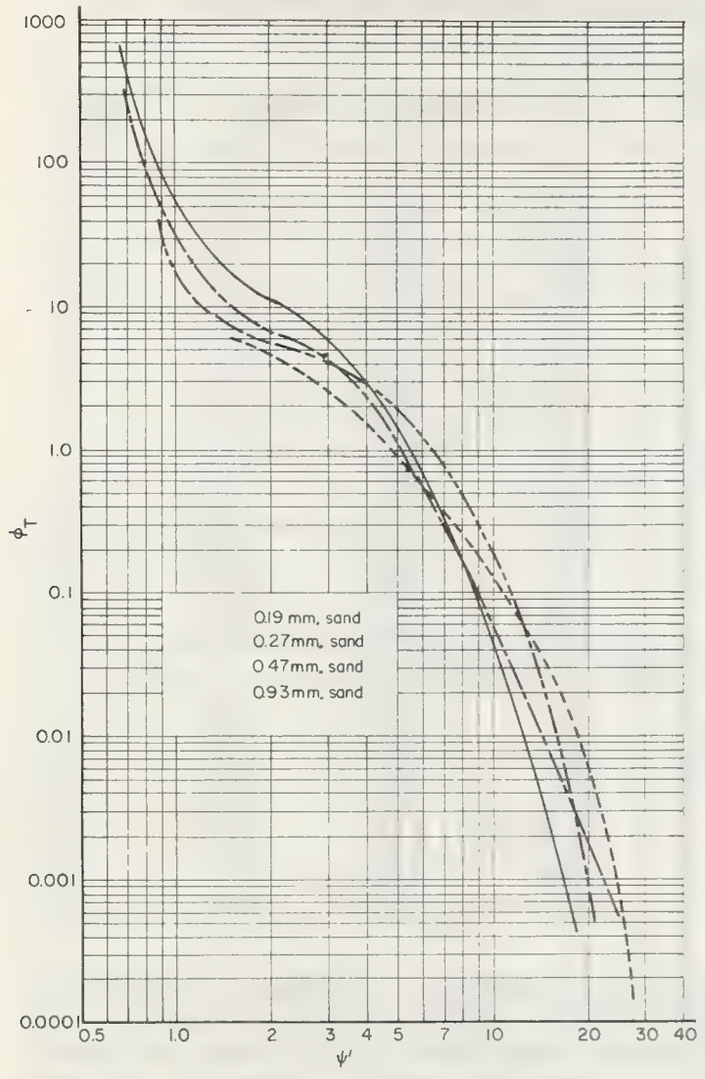


FIGURE 12. — Composite ϕ_T-ψ' curves for various sands from flume data.

Symbols and Units

Symbol	Definition	Units
A_*	Constant for ϕ_T	...
B_*	Constant for ψ_*	...
$C/\sqrt{g}$	Chezy coefficient	...
D	Water depth	Feet
$\triangle D$	Correction to depth	Feet
D'	Effective depth $(D-\triangle D)$	Feet
d_{50}	Size of bed material for which 50 percent is finer, etc.	Feet
f_b	Friction factor of the bed	...
F_r	Froude number	...
i_b	Fraction of the bed material in a given size range.	...
i_B	Fraction of bedload in a given grain size.	...
q_s	Corresponding suspended load rate.	Pounds per second per foot
q_T	Total sediment discharge	Tons per day
R	Hydraulic radius	Feet
R'	Hydraulic radius with respect to the grain.	Feet
R''	Hydraulic radius with respect to channel irregularities.	Feet
S	Slope	...
V	Average velocity	Feet per second
V'	Effective velocity	Feet per second
V_*	Shear velocity $\sqrt{g\,DS}$	Feet per second
V'_*	Effective shear velocity	Feet per second
w	Width of channel	Feet
ρ_s	Density of fluid	Slugs per cubic foot
ρ_f	Density of solids	Slugs per cubic foot
τ	Shear stress	Pounds per square foot
ϕ	Intensity of sediment transport.	...
ϕ_*	Intensity of transport for individual grain size.	...
ϕ_T	Intensity of transport for total bed material load.	...
Ψ'	Intensity of shear on representative particle.	...
Ψ_*	Intensity of shear for individual grain.	...

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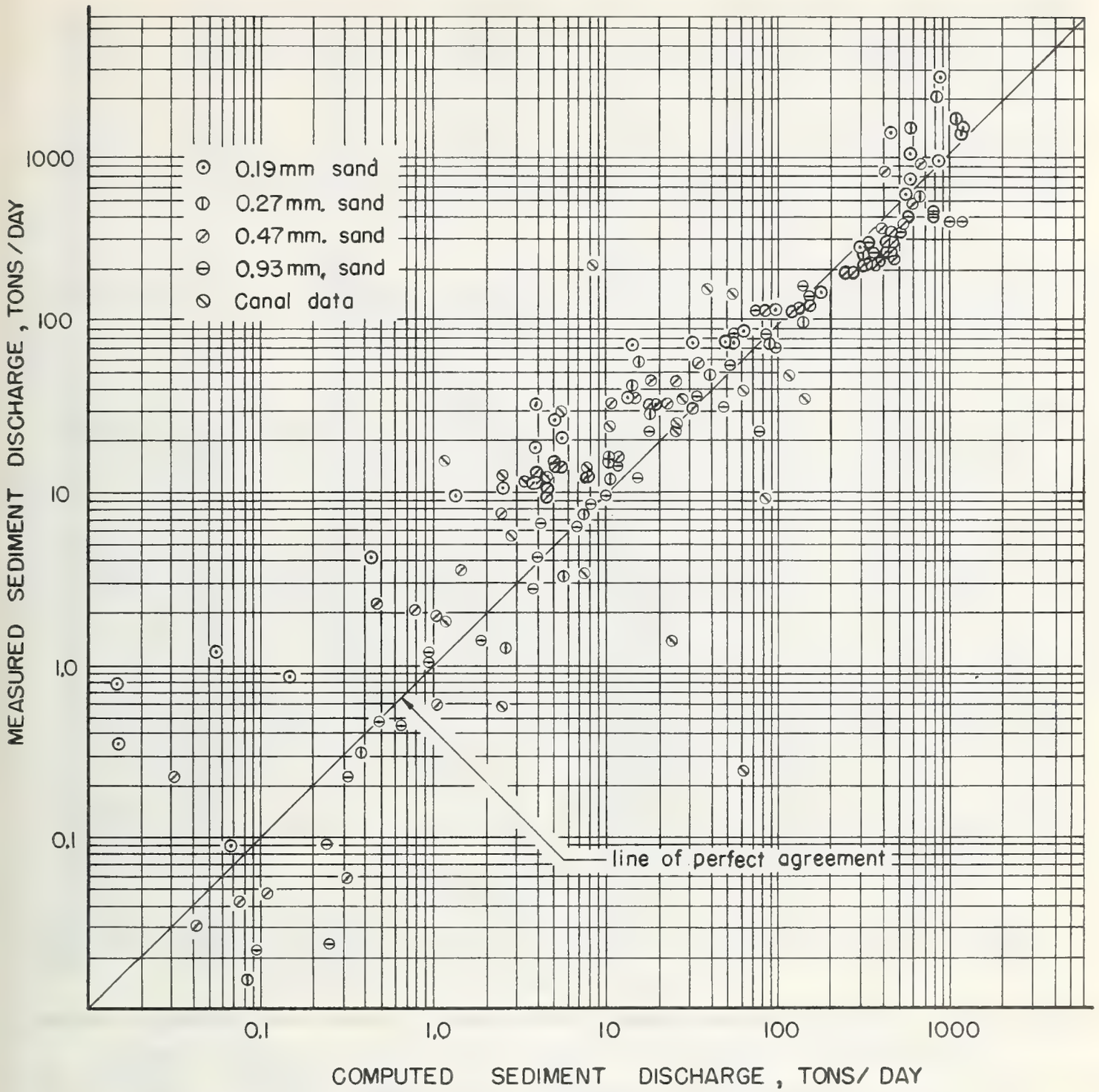

FIGURE 13. — Values of A_* and B_* for theoretical probability curves from laboratory data.

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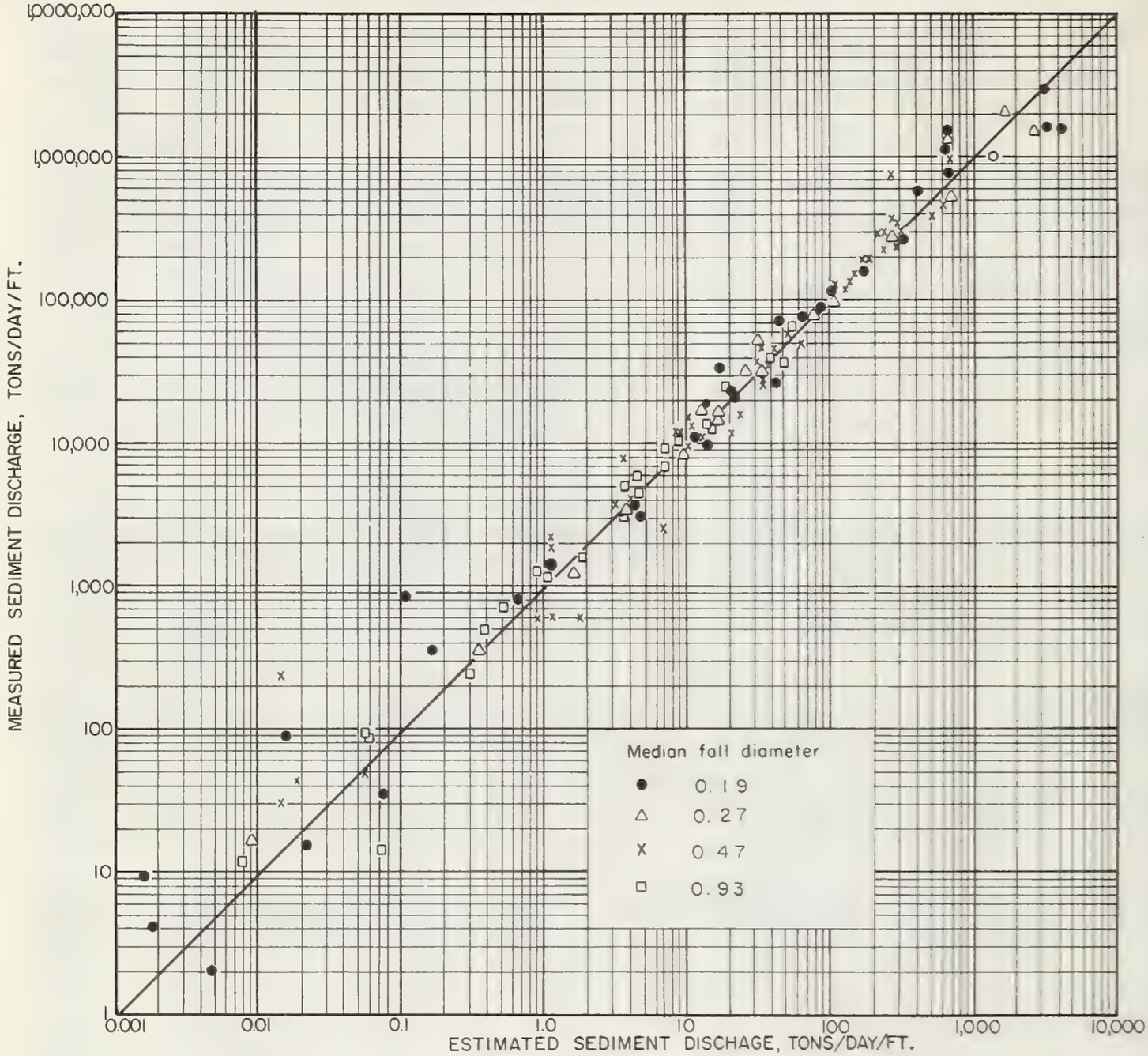

FIGURE 14. — Measured sediment discharge compared to sediment discharge computed using modified ϕ_T-ψ' relation.

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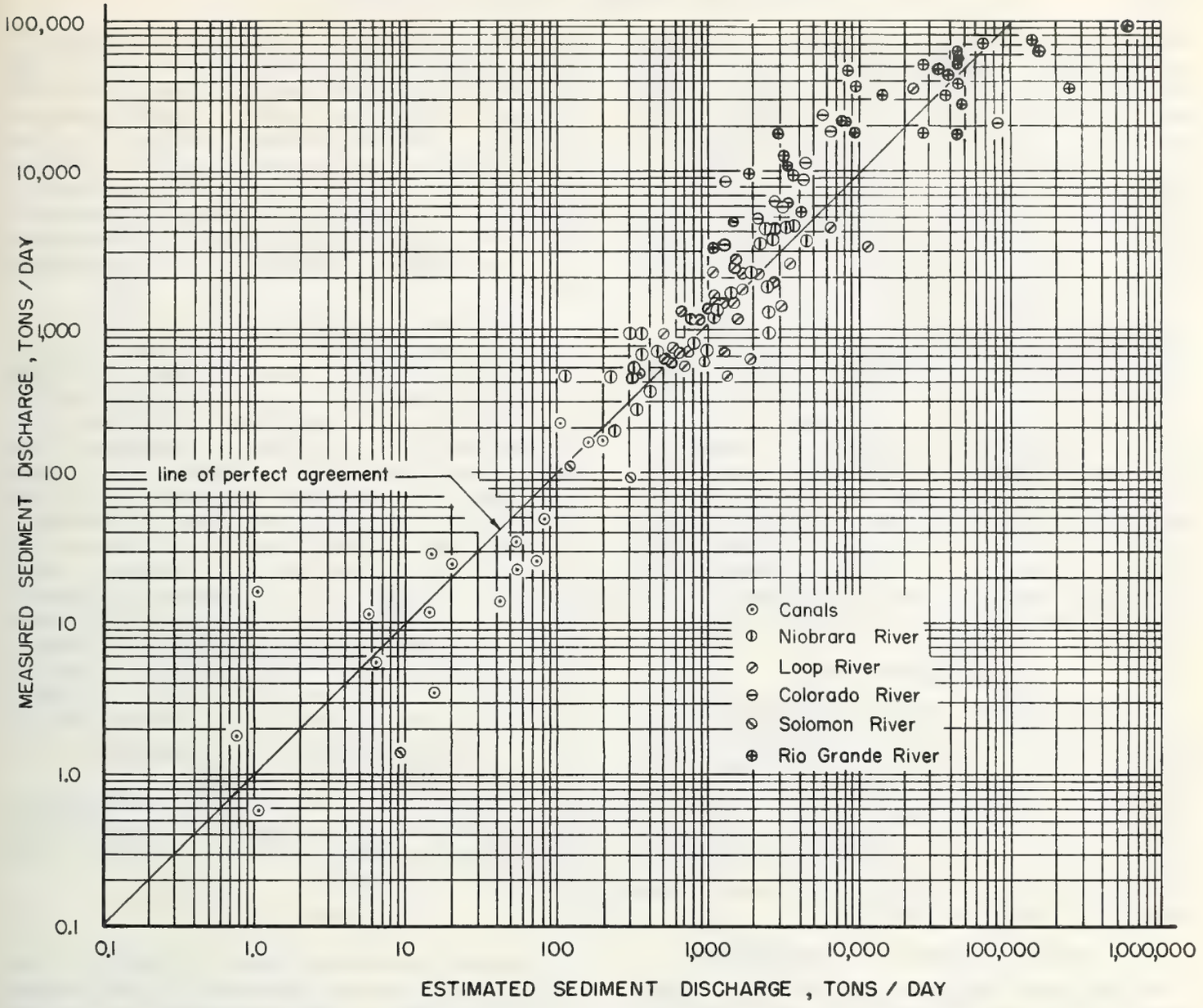


FIGURE 15.—Comparison of bed material load estimated from laboratory curves with measured bed material load.

DEEP RIVER VELOCITY AND SEDIMENT PROFILES AND THE SUSPENDED SAND LOAD

[Paper No. 28]

By FRED B. TOFFALETI, *U.S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Miss.*

Synopsis

A large number of velocity measurements have been made in the Atchafalaya River at Simmesport and in the Mississippi River at Vicksburg to define the vertical velocity distribution. Examination of these data shows the average vertical velocity profile of the two streams to be almost identical at the respective locations. However, a comparison of this measured vertical velocity distribution with that derived by a formula in common usage in analytical sedimentation studies shows these two to be quite different.

Suspended sediment measurements are also made at regular intervals in the Atchafalaya River at Simmesport. Sampling procedure is in

accordance with the Luby method for point measurement. An evaluation of the accuracy of this method is made by a study of 11 sets of measurements covering a wide range of flow conditions. The study indicates that the silt and clay portion of the suspended load was satisfactorily measured, but that there was a significant deficiency in measurement of the sand portion. A detailed study of the fine sand fraction of the suspended load in the 11 examples indicated that the Luby sampling procedure gave fine sand loads averaging about 30 percent less than the apparent actual load. In some instances a part of this deficiency is attributable to abnormal or atypical samples, but even so, economic reasons forbid an adequate number of Luby point samples to measure accurately the sand load in large, deep rivers. The procedure used herein for evaluating the accuracy of sediment measurements would be useful in routine analyses for detecting abnormal or apparently erroneous samples and to approximate the quantity of unmeasured load below the lowermost Luby measuring point.

Introduction

In recent years a considerable number of velocity and suspended sediment measurements have been made in the Atchafalaya and Lower Mississippi Rivers, two large and deep streams in the Lower Mississippi Valley. This paper presents the results of some of these measurements and a token comparison with theoretical and empirical procedures developed to provide this information analytically.

Vertical Velocity Distribution

In the period 1955 through 1960 several hundred vertical velocity profiles were measured in the Atchafalaya River at the Simmesport discharge range (fig. 1). Similar measurements were made in the Mississippi River at the Vicksburg discharge range in the period 1949 through 1956. The results of these measurements in terms of velocity coefficients, or the relation of the mean velocity in the vertical to that at any given depth, are shown in figure 2 and table 1. These data indicate that the average vertical velocity profiles of the two streams are practically identical at the respective locations.

TABLE 1.—*Average velocity coefficient* U/U_y

Proportionate depth	Mississippi River at Vicksburg[1]	Atchafalaya River at Simmesport[2]
[3]*S*	0.915	0.917
0.1	.902	.906
.2	.905	.912
.3	.914	.922
.4	.931	.938
.5	.957	.960
.6	.994	.991
.7	1.045	1.042
.8	1.121	1.120
.9	1.244	1.233
[4]*B*	1.461	

[1] Determined from 500 measurements.
[2] Determined from 473 measurements.
[3] Surface measurement taken at 0.5-ft. depth.
[4] Bottom measurement taken at 1.8-ft. above bottom.

These data were obtained over a period of several years in all seasons and include measurements over a wide range of stage, discharge, and water temperature. A grouping of the data in accordance with conditions of similarity, i.e., season and stage, provided no evidence of distinguishing characteristics attributable to these variables. This approach indicated that the natural scatter or variations due to turbulence obscured the discernment of any seasonal or other effects. Table 2 shows the degree of variation in the 500 sets of Vicksburg measurements by deviations from the average for each tenth of depth. In this tabulation the scatter is more pronounced in the upper and lower regions of flow, particularly the latter. As will be seen later, this greater scatter at the extremities of the velocity profile apparently affects sediment sampling in these areas.

A log-log plot of the velocity coefficients in table 1 applied to any value of mean velocity gives the reasonably accurate vertical velocity distribution formula

$$U_y = 1.15\,U\left(\frac{y}{d}\right)^{0.155} \qquad (1)$$

in which y is distance above the bed, d is the depth of a mean-depth section, U is the average velocity of flow in the mean-depth section, and U_y is the point velocity at y distance from the bed. This velocity distribution for any values of U and d shows that m, the number of velocity units per cycle of 10 of either y or y/d, can be expressed as

$$m = \frac{U}{3.40} \qquad (2)$$

The Von Karman coefficient expressed as $k = \frac{2.3u_*}{m}$ then becomes

$$k = \frac{44.4\sqrt{Sd}}{U} \qquad (3)$$

in which S is the water-surface slope and u_*, the shear velocity, equals $\sqrt{Sdg}$, g being the acceleration due to gravity.

A comparison of the vertical distribution given by formula 1 above with that from formula 3 in USDA Technical Bulletin 1026 [1] shows the two to be quite different. USDA formula 3 gives velocities ranging from about 40 per-

[1] H. A. EINSTEIN. THE BED-LOAD FUNCTION FOR SEDIMENT TRANSPORTATION IN OPEN CHANNEL FLOWS. U.S. Dept. Agr. Tech. Bul. 1026. 1950.

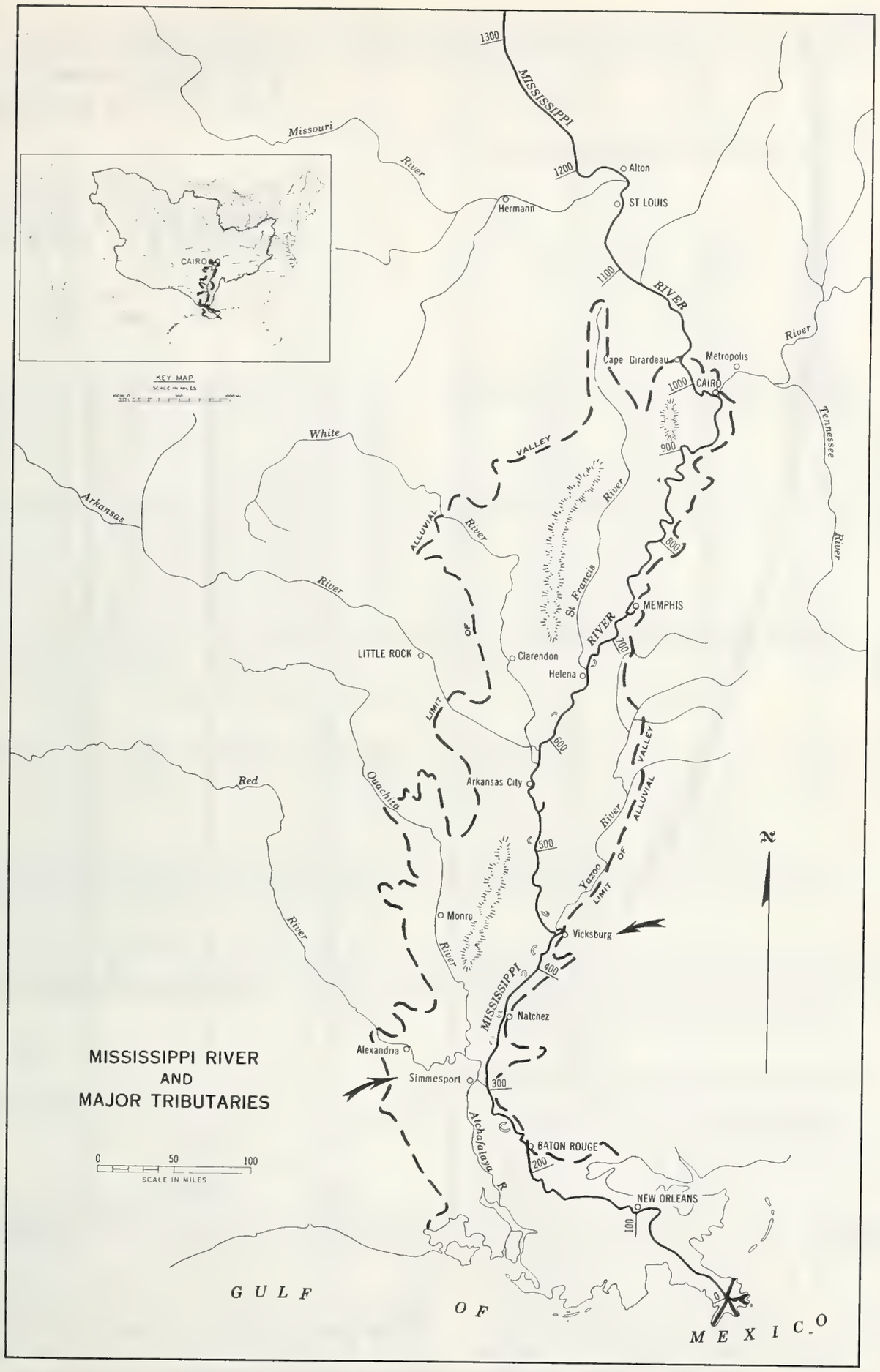


FIGURE 1. — Location of the Simmesport and Vicksburg stream gaging stations.

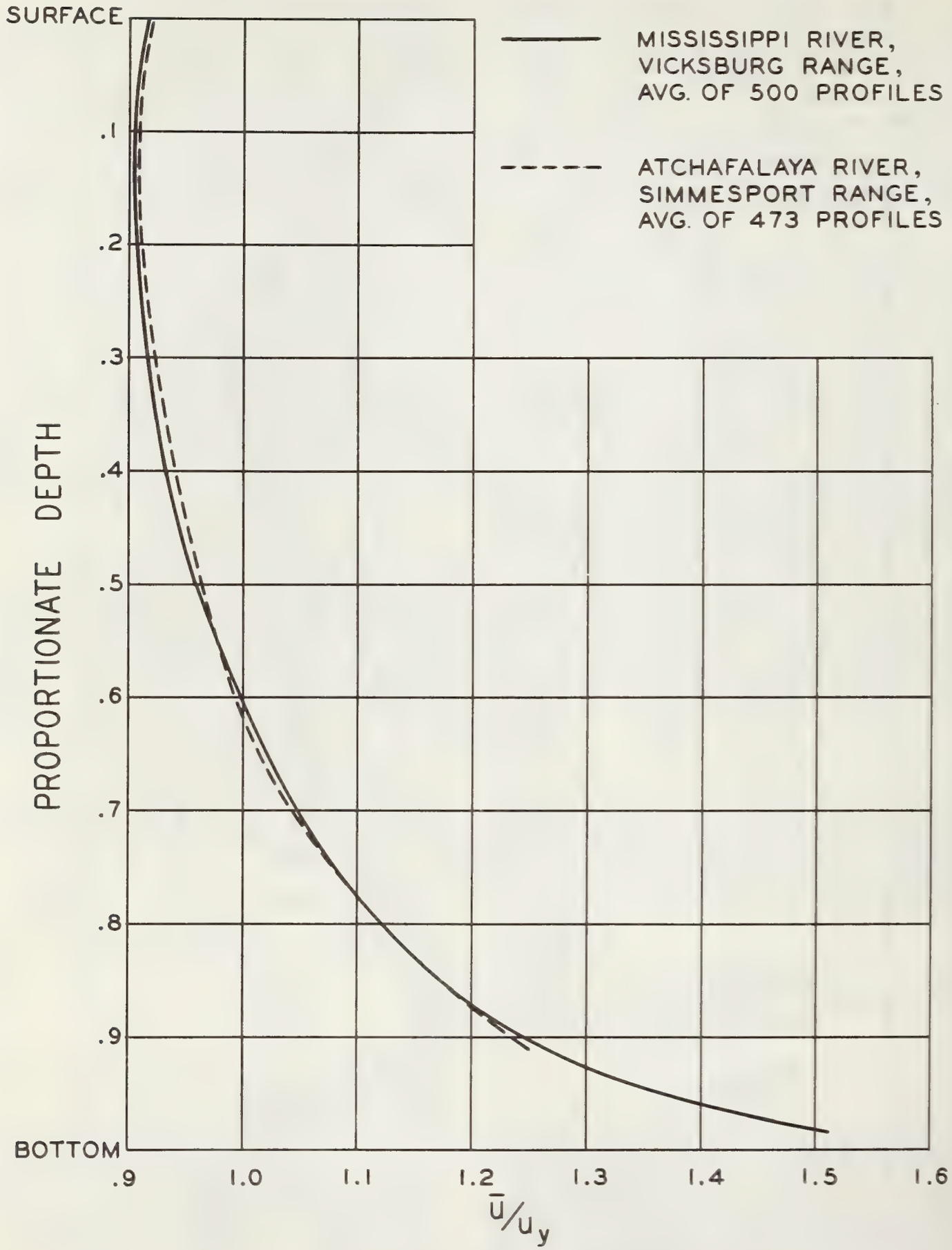


FIGURE 2. — Velocity profiles at Simmesport and Vicksburg ranges.

TABLE 2.—*Summary of deviations from the average velocity coefficient at each tenth of depth for the 500 measurements in the Mississippi River at Vicksburg*

Percent deviation from average	Proportionate depth										
	S [1]	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	B [2]
−20>−20										4	44
−19											6
−18										2	9
−17	1									1	7
−16											15
−15	2							1	3	3	9
−14	1	2						1		7	11
−13	2		1					1	3	7	11
−12	3								5	18	15
−11	5	2	1				1	5	3	7	13
−10	10	1	1				2	3	6	22	13
−9	14	5	2	2		1		4	8	17	10
−8	18	10	3	2		2	4	8	11	13	16
−7	20	17	4	7	2	4	3	14	15	15	10
−6	26	24	14	6	6	9	10	13	23	26	12
−5	29	41	28	17	12	10	22	13	37	20	15
−4	34	37	38	25	16	18	31	37	29	18	11
−3	36	35	50	49	33	32	30	37	32	23	18
−2	26	44	53	72	60	57	59	39	27	26	16
−1	31	33	52	58	77	80	55	45	41	27	22
0	19	35	48	55	67	77	58	44	29	33	19
+1	29	30	43	53	81	76	76	53	34	29	13
+2	35	38	32	54	61	55	54	54	33	22	14
+3	28	23	26	34	35	33	34	39	37	15	11
+4	17	31	29	28	20	25	23	28	28	19	9
+5	24	20	28	15	12	11	15	24	24	25	16
+6	17	24	17	7	6	4	13	15	24	12	12
+7	18	16	12	8	5	4	4	8	12	12	9
+8	13	12	7	3	2	2		8	10	10	13
+9	7	6	3	2	1		2	1	8	9	11
+10	9	3	4		4		1	2	6	15	5
+11	7	5	2				1		1	12	7
+12	5	2	1	1			1	1	4	6	6
+13	2	1					1			2	10
+14	2	1		2				2	2	3	2
+15	1	1							2	5	6
+16	1										3
+17									2	4	4
+18	2									1	5
+19	1								1		2
+20>+20	5	1	1							10	40

[1] Surface measurement taken at 0.5 ft. depth.
[2] Bottom measurement taken at 1.8 ft. above bottom.

cent higher at the surface to about 100 percent higher near the bed. In this comparison the Technical Bulletin 1026 formula 3 was used in the form

$$\frac{U_y}{u_*} = 8.50 + 5.75 \log_{10}\left(\frac{y}{\Delta}\right)$$

If this formula is stated as

$$\frac{U_y}{u_*} = C + 5.75 \log_{10}\left(\frac{y}{\Delta}\right)$$

it is found that to reproduce the vertical velocity distribution determined by formula 1 would require C to be a negative number. Further, C would not be a constant range of y values, bottom to surface.

Although the procedure outlined in Technical Bulletin 1026 for determining sediment loads has previously been used in Atchafalaya River studies, the preceding comparison of formulas indicates that use of the vertical velocity distribution formula 3 for sediment studies of either the Atchafalaya or Lower Mississippi River would give loads greatly in excess of measured loads.

Sediment Sampling and the Total Load

Suspended sediment sampling was initiated in 1949 on the Lower Mississippi River at Baton Rouge, and in 1951 on the Atchafalaya River at Simmesport. In 1958 the Baton Rouge operation was transferred to Red River Landing, about 72 miles upstream. Sampling is generally at 2-week intervals, more frequent during floods when possible, and less frequent during the low-water season. The Luby method for point sampling was adopted for these stations, with 40 samples taken in Mississippi River cross sections, a sample from 5 levels in each of 8 verticals, and 25 samples in the Atchafalaya

River at 5 levels in 5 verticals. Bed material samples are also taken at each river station at each vertical of suspended sediment measurement.

Over the period of observations, which included all types of water years, the measured suspended sediment in the Lower Mississippi River averaged 84 percent finer than 62 microns at Baton Rouge and 71 percent at Red River Landing. In the Atchafalaya River at Simmesport the average was 76 percent. Bed material samples from the Baton Rouge and Simmesport stations show the dominant material to be fine and very fine sand, and very little of the sand portion of the measured suspended sediment at these stations exceeded ¼ mm. in size. The characteristics of these suspended sediments, then, are such that the Luby five-point method would be expected to give an overall total-load accuracy within 1 or 2 percent. A summary of the results of observations at the three stations discussed above is shown in table 3.

Evaluation of Measured Load

The discussion that follows attempts to evaluate the accuracy of suspended sediment measurements and load determinations of the Atchafalaya River at Simmesport.

The laboratory analyses of the suspended sediment samples indicate generally that the material finer than 62 microns is fairly evenly distributed in the vertical, and that a major portion of this fraction of the load is in the size range of very fine silts and clays. As these fine materials comprise about 76 percent of the total load on a long-term basis, it is considered that this major portion of the total load is accurately measured. For an estimate of accuracy of measurement of the suspended sands, the fine sand fraction (¼ to ⅛ mm.) was selected for study. This is generally the dominant bed material and is found in transport at all levels in the channel section except at extremely low flows.

Figure 3, *A* to *K*, presents 11 examples of fine sand distribution as determined from analyses of the point samples. These examples cover a wide range of discharge with water depths at the verticals varying from 83 to 46 ft. on the deep side of the channel, and from 54 to 26 ft. on the shallow side. The figures show measured data for six levels in the verticals. The five measurements and average thereof on the extreme right in each figure show the results of measurements made at 3 ft. from the bed in each vertical. This is a measurement somewhat below the lowest Luby point in each case and is not used in load computation. Its purpose is to attempt to define sediment distri-

TABLE 3.—*Summary of measured suspended sediment loads* [1]

ATCHAFALAYA RIVER AT SIMMESPORT, LA.

Water year (Oct.-Sept.)	Total measured sediment load (in 1,000 tons)	Sand-silt ratio				Water-year discharge (1,000 d.s.f.)	Average sediment concentration (p.p.m.)
		Sand		Silt			
		In 1,000 tons	Percent	In 1,000 tons	Percent		
1951–52	196,460	48,890	25	147,570	75	80,800	900
1952–53	135,230	28,440	21	106,790	79	56,960	880
1953–54	54,130	13,110	24	41,020	76	31,980	627
1954–55	93,360	24,080	26	69,280	74	50,425	686
1955–56	67,175	15,450	23	51,730	77	49,080	507
1956–57	225,474	55,700	25	169,774	75	74,059	1,126
1957–58	214,390	48,082	22	166,308	78	89,413	887
1958–59	83,230	20,944	25	62,286	75	55,729	553
1959–60	131,878	24,153	18	107,725	82	69,333	704
1960–61	133,372	40,524	30	92,848	70	76,814	643
LOWER MISSISSIPPI RIVER AT BATON ROUGE AND RED RIVER LANDING, LA. [2]							
1949–50	548,330	107,770	20	440,560	80	245,200	828
1950–51	575,280	67,600	12	507,680	88	224,810	947
1951–52	408,390	73,820	18	334,570	82	200,660	754
1952–53	212,580	28,920	14	183,660	86	142,200	552
1953–54	107,730	14,090	13	93,650	87	88,660	449
1954–55	211,490	39,930	19	171,550	81	137,460	570
1955–56	161,220	25,920	16	135,300	84	127,530	468
1956–57	291,388	53,043	18	238,345	82	172,875	624
1957–58	325,774	95,203	29	230,571	71	195,653	616
1958–59	230,504	78,693	34	151,811	66	129,253	660
1959–60	318,234	77,219	24	241,015	76	163,850	718
1960–61	231,754	71,471	31	160,283	69	168,133	510

[1] The sand fraction is the material retained on the No. 230 sieve (0.062 mm.). The silt fraction includes all of the fine material passing the No. 230 sieve.

[2] Sampling at Red River Landing after 1 January 1958.

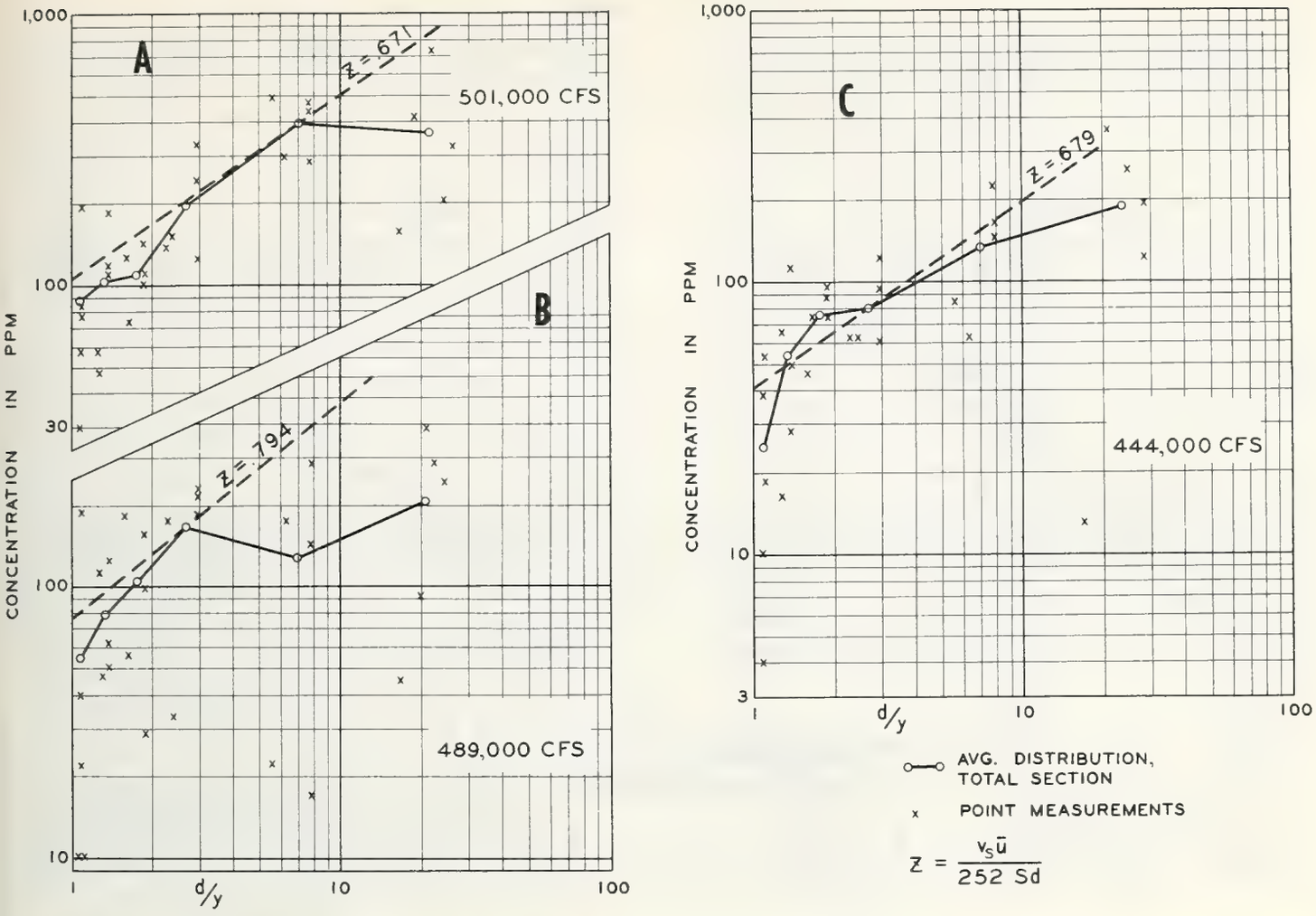


FIGURE 3. — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *A*, for 501,000 c.f.s.; *B*, for 489,000 c.f.s.; *C*, for 444,000 c.f.s.

bution nearer the bed than dictated by the 5-point Luby method. Although, for the purpose stated, it would be desirable to obtain samples closer to the bed than 3 ft., experience in sampling in deep turbulent flow has shown this to be the lowest practicable sampling level from considerations of possible loss or damage to instruments. The plot in these figures is concentration versus d/y rather than $\frac{d-y}{y}$, a more common procedure. This was done for convenience of load computation, but it was also found that the vertical distribution could be defined by d/y as well as, or better than, by the conventional plot. The distribution is not satisfactorily definable by a single line in either case.

The plot of measured points in figure 3 shows a wide scatter in fine sand concentration values at all levels in each example, but in most of the examples the scatter appears to be somewhat less at and near middepth. The reason for this may be due to fluctations in the velocity pattern as indicated in table 2. The velocity coefficient is relatively stable at and near middepth, but fluctuates to an increasing degree toward each extremity of the vertical profile.

In order to better define the vertical distribution of fine sand, each common level of measurement across the channel was averaged. In essence, this produced an average vertical distribution profile in a mean depth channel section. For comparison with the measured average vertical distribution, a theoretical vertical distribution of fine sand is indicated by the dashed line in each example. The evaluation of the coefficient Z for this theoretical distribution was determined by the expression

$$Z = \frac{v_s}{ku_*} = \frac{v_s}{k\sqrt{Sdg}} \qquad (4)$$

in which v_s is the settling velocity of fine sand grains in feet per second. By substitution for k from formula 3

$$Z = \frac{v_s U}{252\ Sd}$$

In about two-thirds of the examples presented, the measured data approximate this theoretical distribution from about middepth to 0.85 depth,

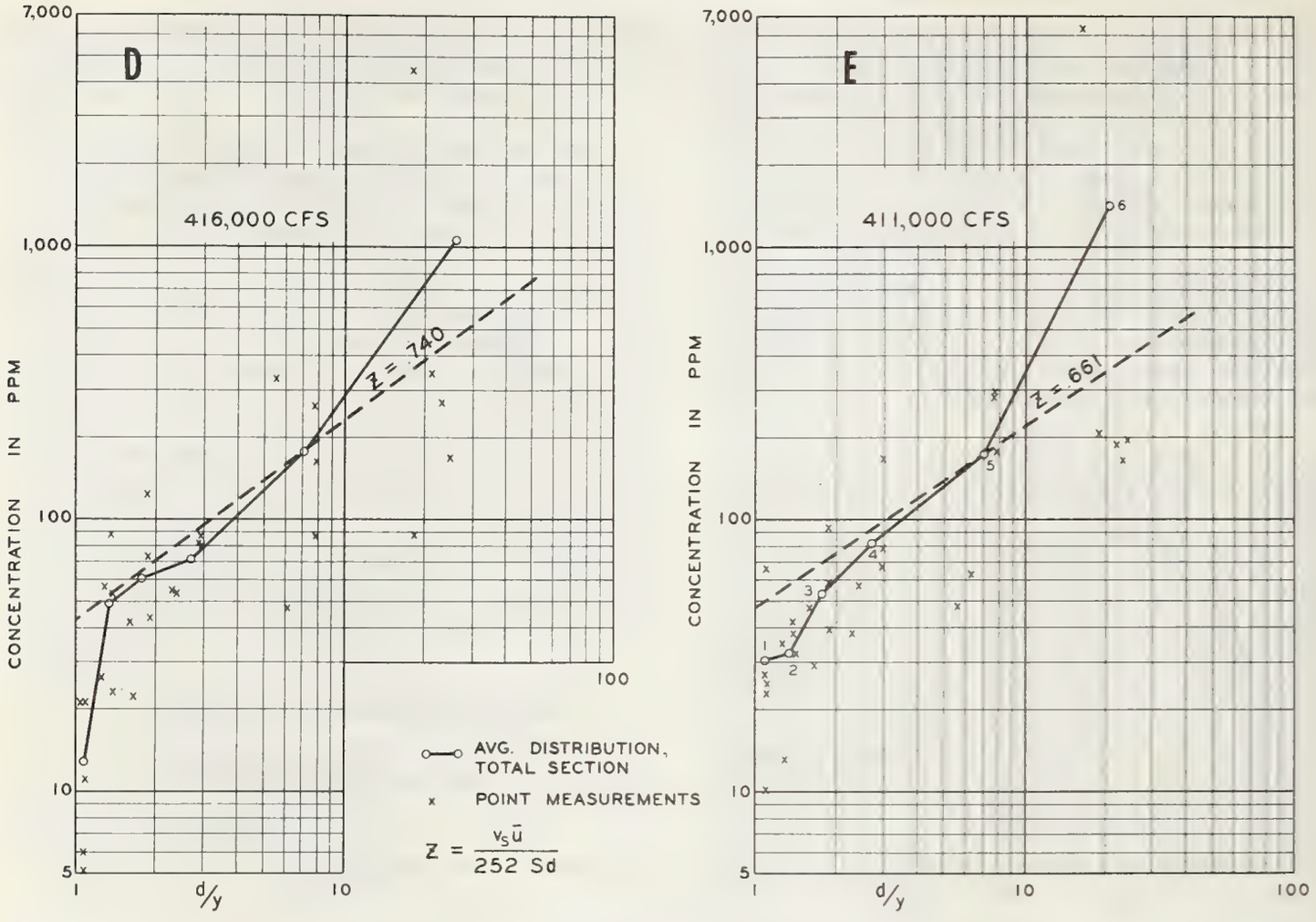


FIGURE 3 (continued). — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *D*, for 416,000 c.f.s.; *E*, for 411,000 c.f.s.

or about one-third of the total depth. Several of the other examples, especially as shown in parts *B* and *K* of figure 3, have apparent sampling errors, or at least the questionable samples are not typical if a logical order of sediment density from surface to bottom is an accepted fact. In this regard it is not readily understood why the concentration at the points 3 ft. above the bed should be generally out of line on the low side. In figure 3, *D*, *E*, and *F* the average is high, but in each of these cases the high average is caused by a single sample of very high concentration.

In evaluating the accuracy of the fine sand measurements, the average profiles of figure 3 were each described by three lines. In fitting these lines to the average profiles it was assumed that the profiles approximated the true sediment distribution within the limits of the Luby points, except in cases such as shown in figure 3, *B* and *K*. One of the lines was fitted to points representing the profile in the approximate upper half of the channel section; another was fitted to profile points from approximate middepth to the level $d/y=11.24$; and a third line extended from $d/y=11.24$ to near bed at $y=3.68D_{65}$. The slope of the line to near bed was set as the equivalent of that of a $\frac{d-y}{y}$ versus concentration plot of the middle line. Any slope or position of the middle line plotted with d/y values will intersect a $\frac{d-y}{y}$ plot at $d/y=11.24$. The lower limit of suspended load was assumed to be at the level $y = 3.68\ D_{65}$. For cases such as shown in figure 3, *B* and *K*, the middle line was assumed to be the dashed line, with extension of this line in the case of figure 3, *K*, to include all distribution from $d/y=11.24$ to $d/y=1$. To illustrate this, in figure 3, *E*, the sediment distribution in the upper portion of the channel section would be described by a line passing midway between points 1 and 2 and through point 3. Between this line and a point on $d/y=11.24$, the distribution would be described by a line passing through points 4 and 5. From a point on this line at $d/y=11.24$, the distribution to the lower limit of suspended material is

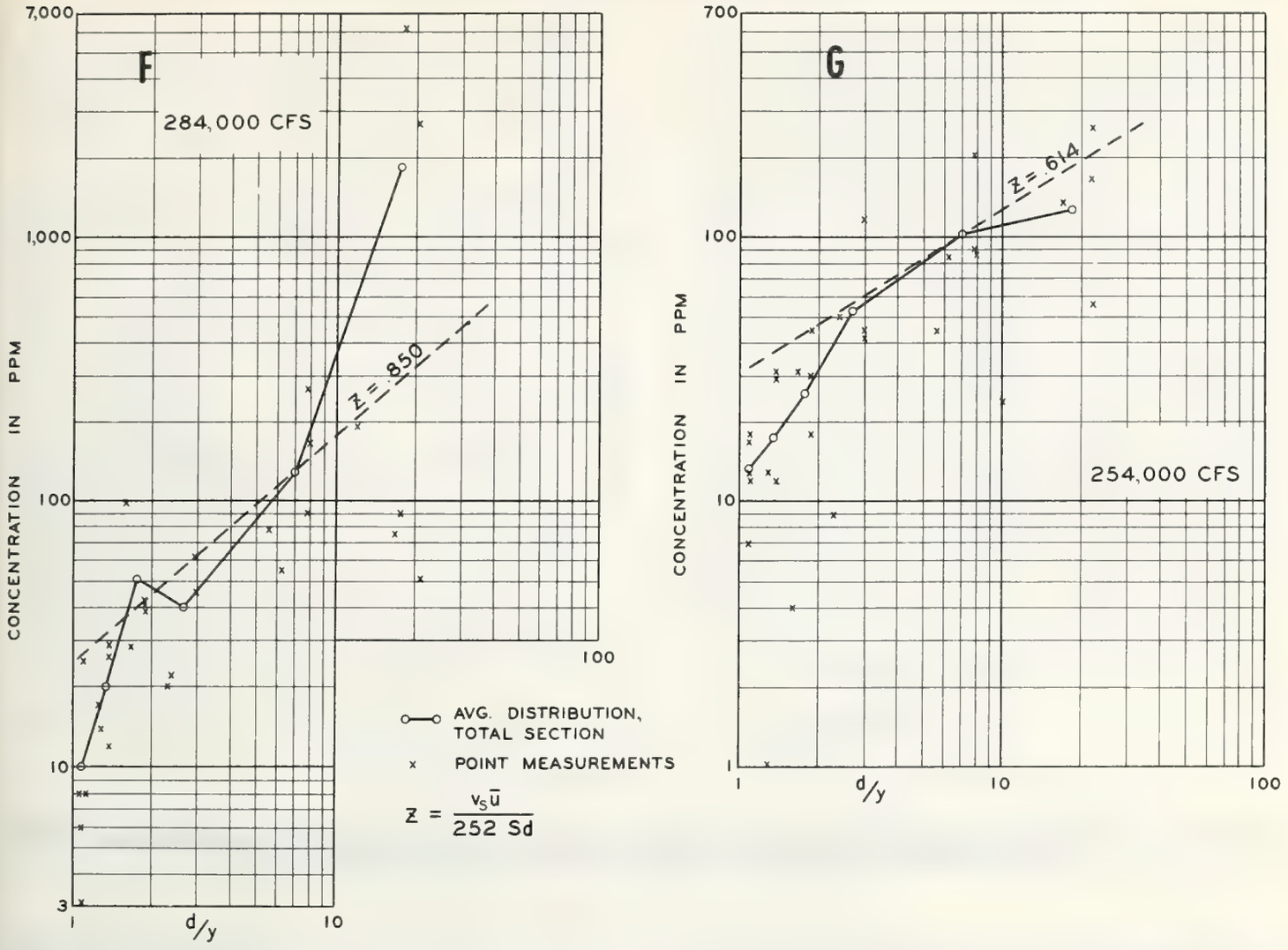


FIGURE 3 (continued). — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *F*, for 284,000 c.f.s.; *G*, for 254,000 c.f.s.

on a line having a slope 0.756 times the slope of the preceding intersecting line. This distribution line near the bed will then be at a slope equivalent to a $\frac{d\text{-}y}{y}$ plot of the line passing through points 4 and 5.

The above-described procedure permits each of the sediment distribution lines to be formulated in terms compatible with formula 1 for vertical velocity distribution. It is then a rather simple mathematical process to compute the total fine sand load. The results of these computations and a comparison with the loads determined by the measurements alone are shown in figures 4 through 14. These graphs show the respective fine sand load in tons per day per 0.10-ft. increments of depth at any level in the vertical over the entire width of a mean depth channel section. This type of plot is of special interest in that indicated loads from abnormal measurements are quite obvious at first glance. Also, it is immediately noticeable that there is a significant deficiency in sand load as determined from the five-point Luby sampling procedure. This deficiency is in the region near the bed, and is due to the relatively great distance of the lowest Luby point from the bed. In the 11 examples presented, the individual bottom samples are taken from 5 to 13 ft. above the bed.

The procedure for computing the suspended fine sand load is shown by the following example:

In figure 3, *E*, between the levels $y=3.68$ $D_{65}=0.0042$ ft. and $d/y=11.24$ or $y=4.3$ ft., the fine sand concentration C_1 at any point is

$$C_1=59\left(\frac{d}{y}\right)^{0.588}=577/y^{0.588}.$$

From level $y=4.3$ ft. to $d/y=2.1$ or $y=23.0$ ft. the fine sand concentration C_2 is

$$C_2=37.5\left(\frac{d}{y}\right)^{0.778}=767/y^{0.778}, \text{ and}$$

from level 23.0 ft. to the surface the fine sand concentration C_3 is

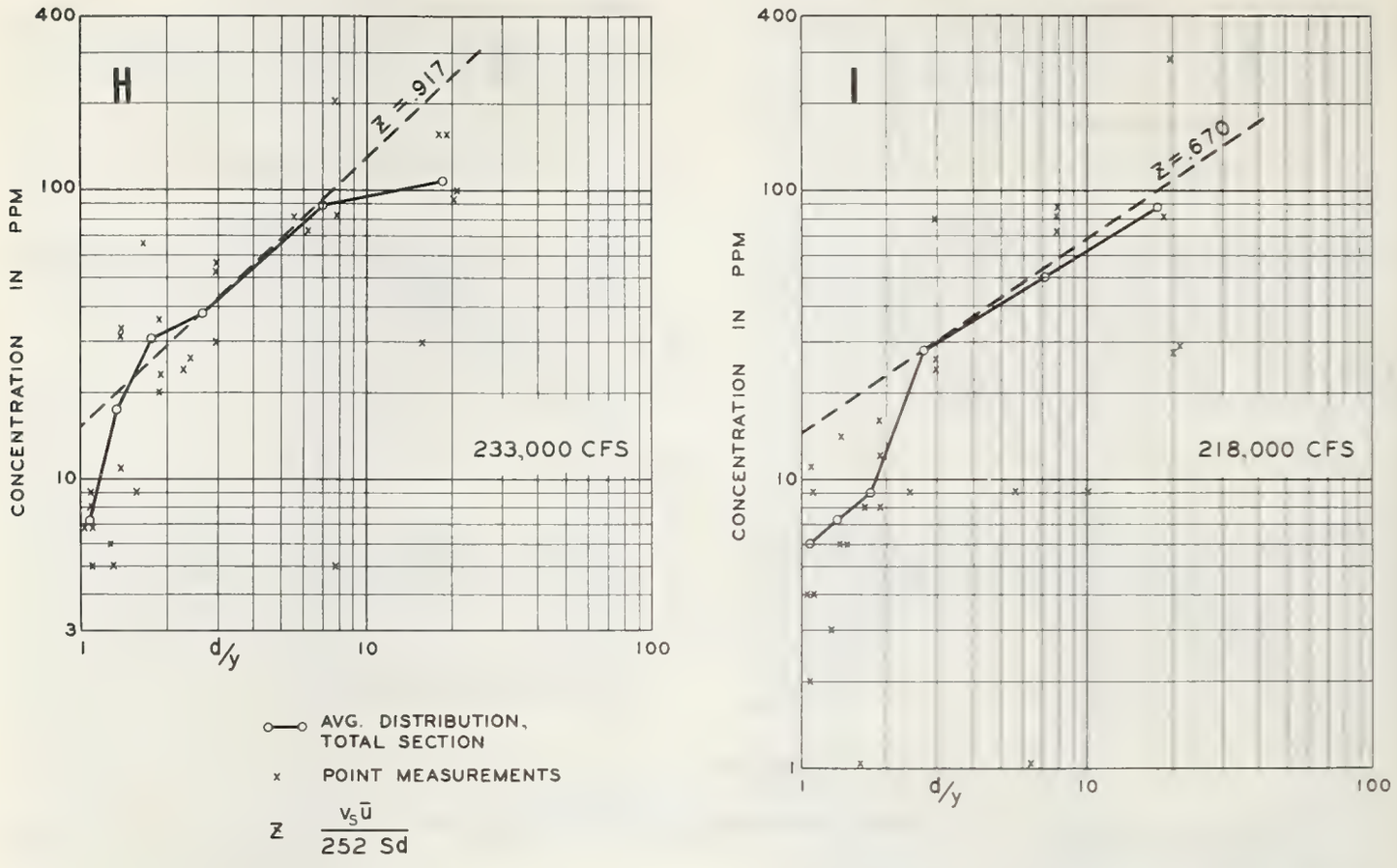


FIGURE 3 (continued). — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *H*, for 233,000 c.f.s.; *I*, for 218,000 c.f.s.

$$C_3=23.8\left(\frac{d}{y}\right)^{1.38}=\frac{23.8\,d^{1.38}}{y^{1.38}}$$

These expressions for distribution of fine sand in parts per million are each multiplied by the velocity distribution formula 1, which for this case is $Uy=3.48\,y^{0.155}$, and then multiplied by 4.146, to convert the distribution to tons per day over the width of channel. The resulting expressions are

$$C_1U_y=\frac{8.325}{y^{0.433}};\ C_2U_y=\frac{11{,}066}{y^{0.623}};\ C_3U_y=\frac{343.4\,d^{1.38}}{y^{1.225}}$$

We can now determine the total or incremental sediment loads within the prescribed limits of each by integration. Resultant expressions are:

$$\text{Sediment load } S_1=14{,}683\,y^{0.567}\Big]\begin{matrix}y=4.3\\ \\ y=0.0042\end{matrix}$$

$$\text{Sediment load } S_2=29{,}353\,y^{0.377}\Big]\begin{matrix}y=23.0\\ \\ y=4.3\end{matrix}$$

$$\text{Sediment load } S_3=134{,}613\left(\frac{d}{y}\right)^{0.225}\Big]\begin{matrix}y=23.0\\ \\ y=48.4\end{matrix}$$

To check the accuracy of fitting the distribution lines to measured data, the computed load above the lowest measured value coincident with the computed load graphs in figures 4 through 14 was added to the load encompassed by the measured load graph below that point. This load in every case was within 3 percent of the total measured load, which indicates a satisfactory alinement of distribution lines above the lowermost Luby point. As the analytical treatment of the load below this level logically assumes that the concentration of sand will increase toward the bed, any significant difference in measured and computed load is generally in that portion of the load between the lowest Luby point and the bed. It is thus indicated in the 11 examples presented that an upward adjustment of the measured fine sand loads in amounts from 8 to 84 percent is warranted. The overall average for the 11 cases is +32 percent. If these examples, which were selected at random, are typical and it is assumed that the total meas-

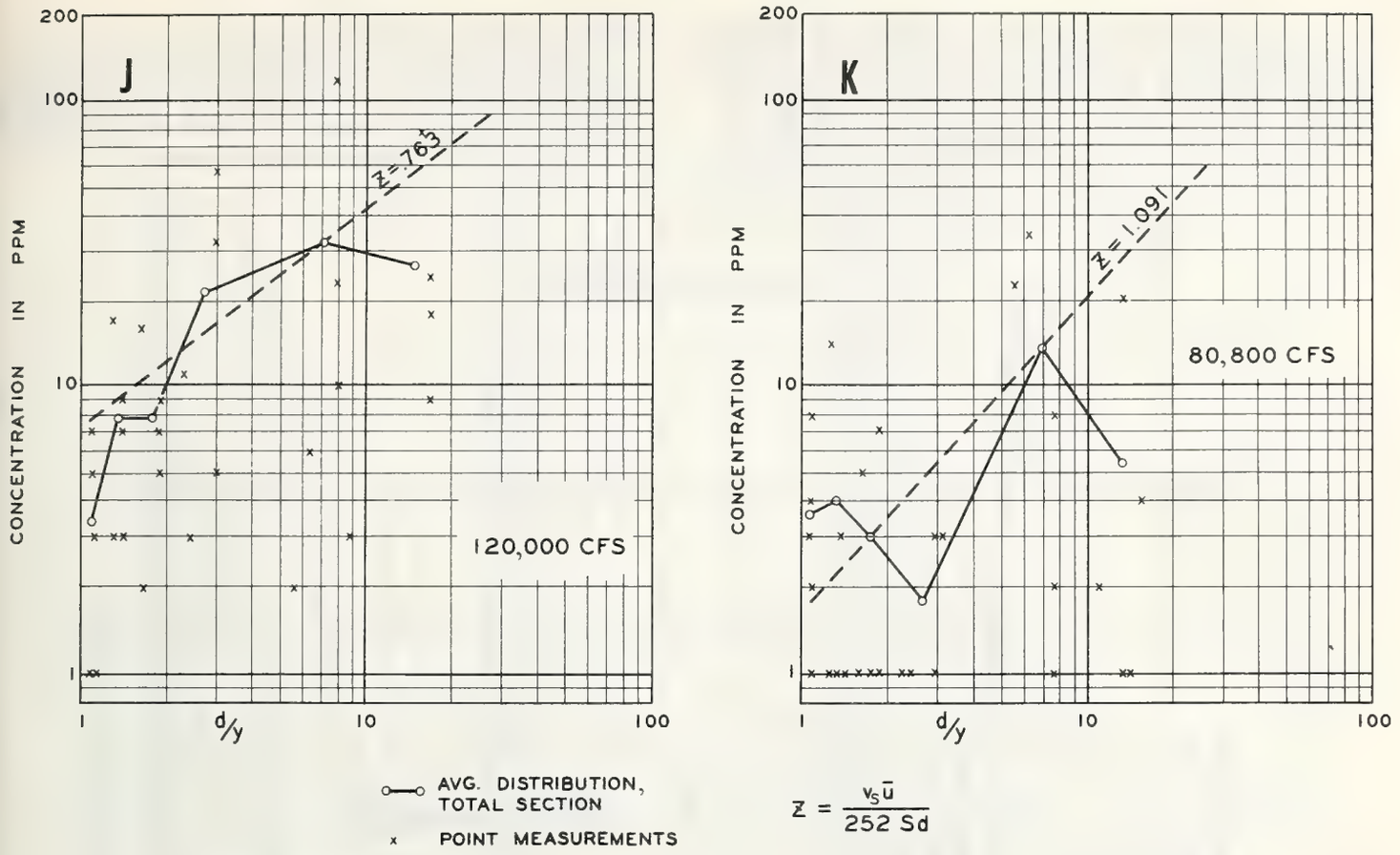


FIGURE 3 (continued).— Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *J*, for 120,000 c.f.s.; and *K*, for 80,800 c.f.s.

ured sand load in suspension is equally deficient, the unmeasured sand load in the 10-year period would total 106 million tons. The examples used in this study could well be typical, and it is reasonable to assume that measurements of all the sand load are equally short.

Discussion

It must be realized that in field operations an abnormal or atypical sample cannot be recognized. Also, when the samples are received in the laboratory a settling period of at least 2 weeks is required before an analysis can be made. Therefore, since it is several weeks after a set of samples is taken before a load determination can be made, averaging is depended on to equalize the deviations. The unreliability of such dependence is exemplified in the measurements at a flow of 489,000 c.f.s., as shown in figs. 3, *B*, and 5. In this case two samples at the lowermost Luby point in two of the five verticals are apparently in error and in sufficient amount to affect greatly the evaluation of measured load. The total sediment content of each of these samples was about one-half of that in the next two samples 13 ft. farther from the bed. The sand contents were 15 and 22 percent, respectively, of those in the next higher sampling level. It is considered that of the 117,774 tons per day difference between the computed and measured fine sand load, about 55,000 tons is attributable to the two abnormal samples.

Conclusions

The examination and analysis of the velocity and sediment profiles selected for presentation in this paper suggest the following conclusions:

(1) The vertical velocity distribution formula 3 in U.S. Department of Agriculture Technical Bulletin 1026 is not applicable to large rivers, such as the Lower Mississippi River and the Atchafalaya River.

(2) In studies involving the velocity and sedimentation characteristics of large rivers, the treatment of entire channel sections as a unit is a satisfactory procedure.

(3) Measurements of the suspended sand load by the Luby method are deficient.

(4) As a supplement to point sampling by the Luby method, additional samples are required near the bed to define sediment transport in that region. A basic weakness in the Luby method

is that, regardless of the number of points measured, more samples will always be taken above middepth than below. Therefore, an excessive total number of samples would be required for adequate sampling below middepth, and the cost would be prohibitive.

(5) The accuracy of point sampling should be evaluated by a plot of data as presented herein, and the measured loads adjusted as necessary to describe the apparent actual load.

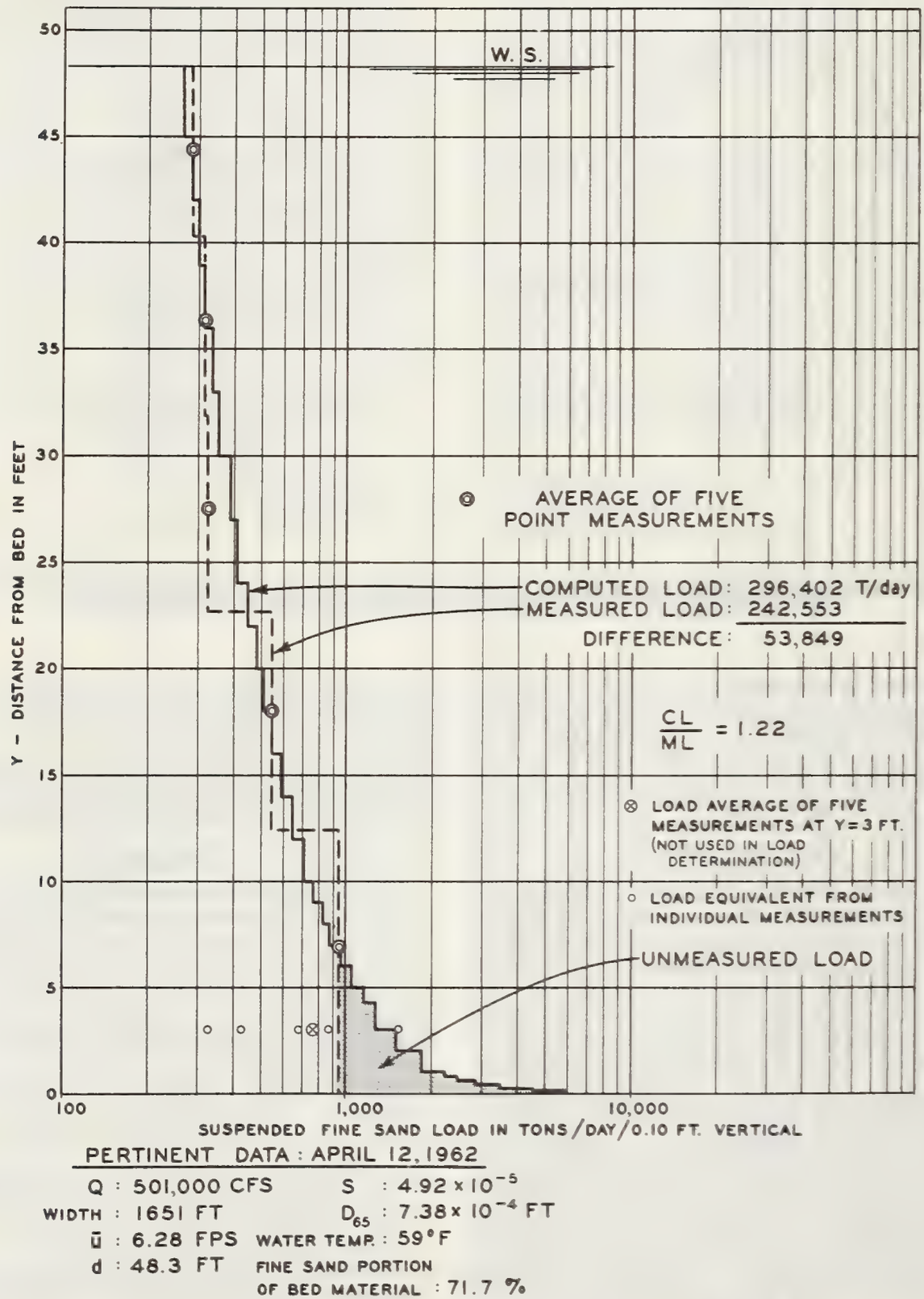
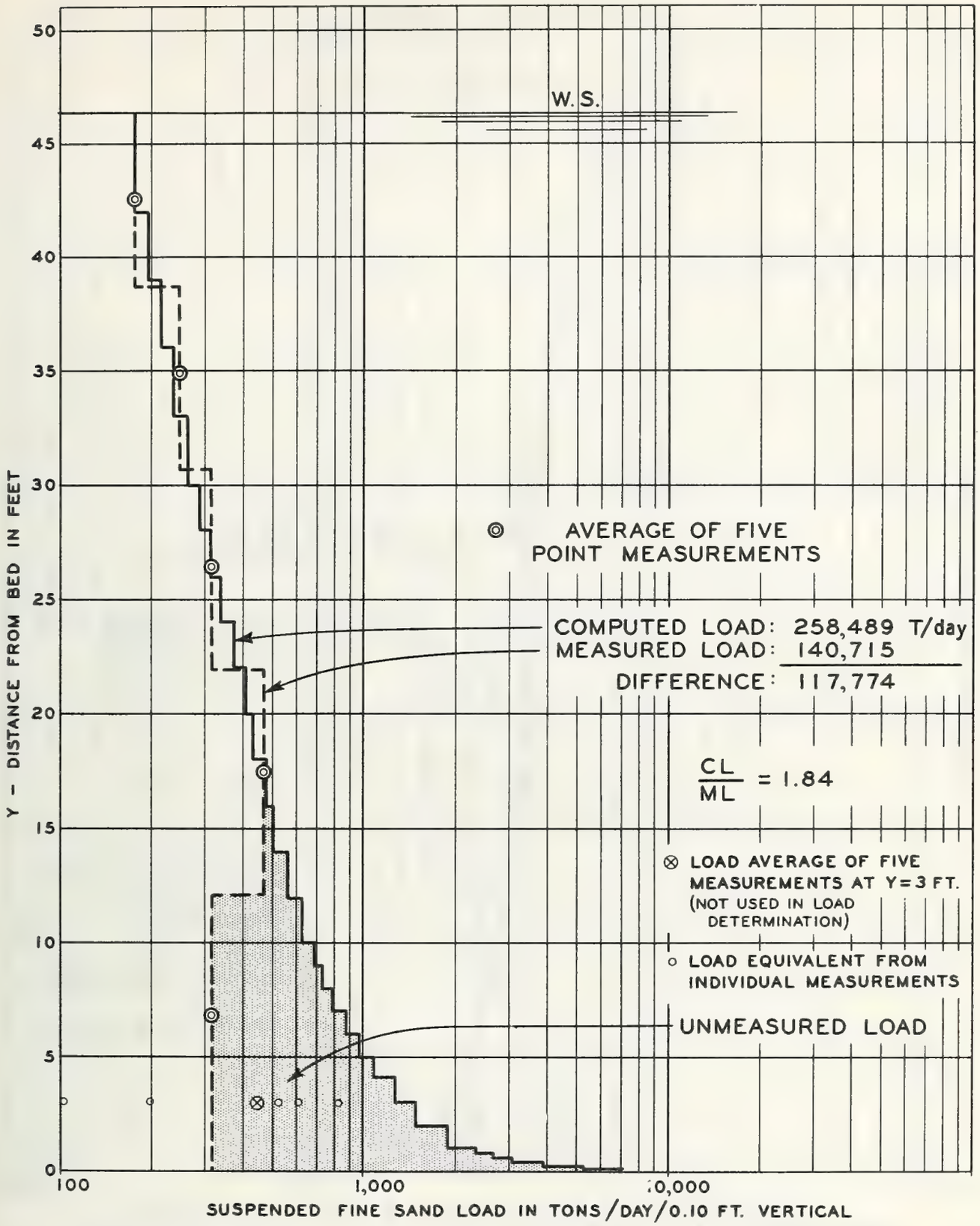

W. S.
AVERAGE OF FIVE POINT MEASUREMENTS
COMPUTED LOAD: 296,402 T/day
MEASURED LOAD: 242,553
DIFFERENCE: 53,849
CL/ML = 1.22
LOAD AVERAGE OF FIVE MEASUREMENTS AT Y=3 FT. (NOT USED IN LOAD DETERMINATION)
LOAD EQUIVALENT FROM INDIVIDUAL MEASUREMENTS
UNMEASURED LOAD
Y - DISTANCE FROM BED IN FEET
SUSPENDED FINE SAND LOAD IN TONS/DAY/0.10 FT. VERTICAL
PERTINENT DATA: APRIL 12, 1962
Q : 501,000 CFS
S : 4.92 x 10⁻⁵
WIDTH : 1651 FT
D65 : 7.38 x 10⁻⁴ FT
ū : 6.28 FPS
WATER TEMP. : 59°F
d : 48.3 FT
FINE SAND PORTION OF BED MATERIAL : 71.7 %

FIGURE 4. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, April 12, 1962.



PERTINENT DATA: JUNE 5, 1961

Q : 489,000 CFS S : 5.03×10^{-5}

WIDTH : 1587 FT D_{65} : 6.75×10^{-4} FT

$\bar{u}$: 6.66 FPS WATER TEMP. : 70°F

d : 46.3 FT FINE SAND PORTION OF BED MATERIAL : 81.6 %

FIGURE 5.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, June 5, 1961.

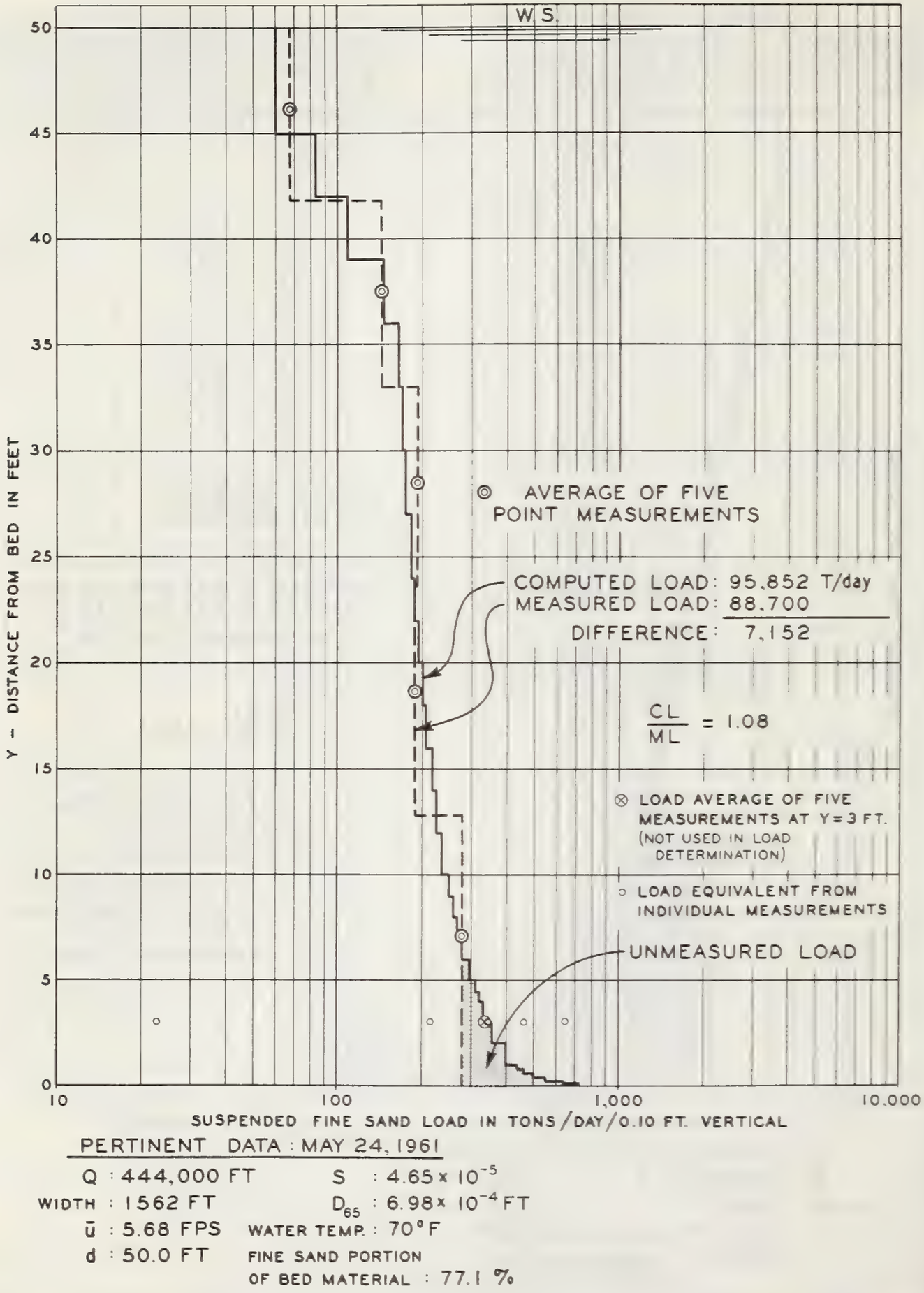
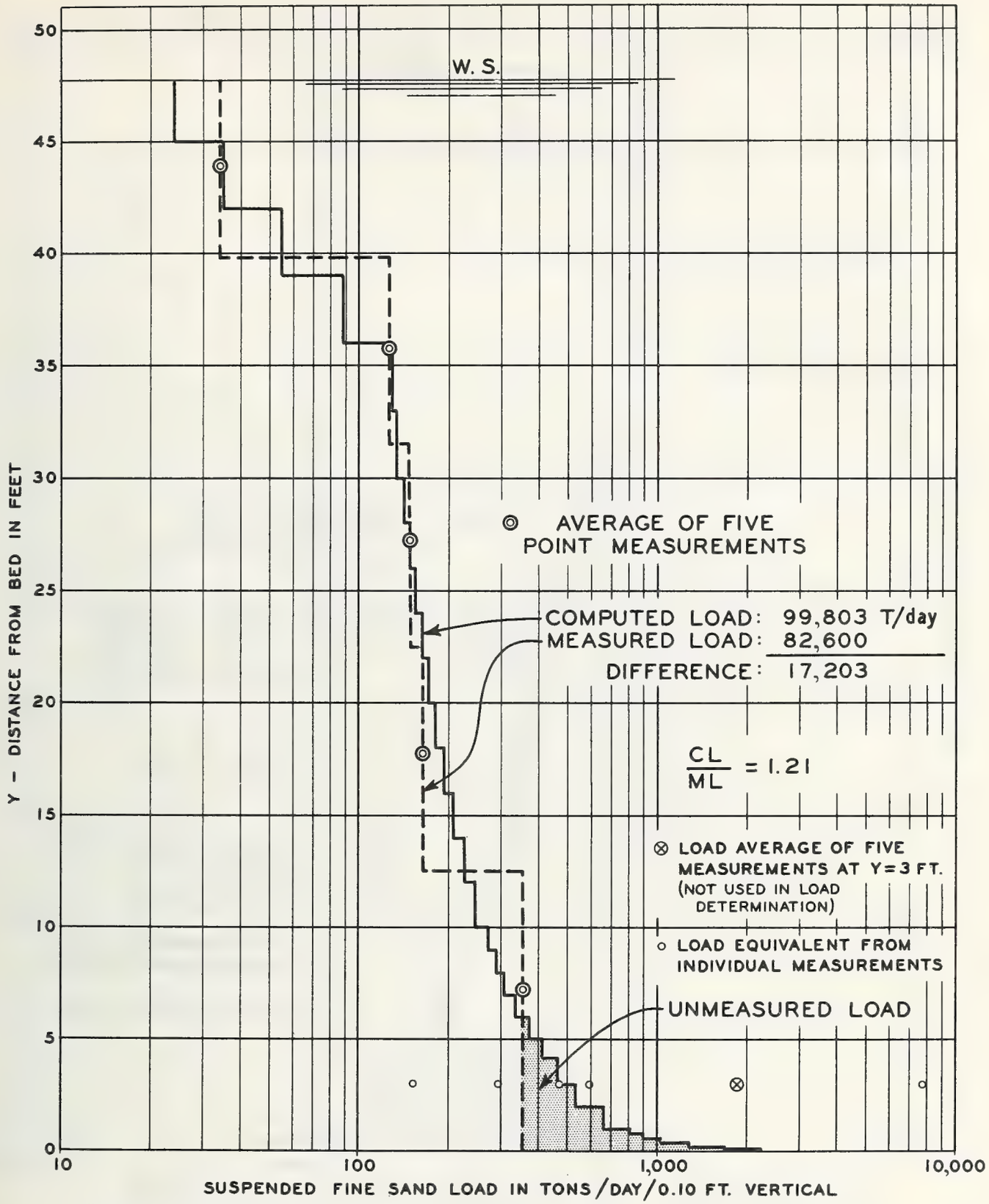


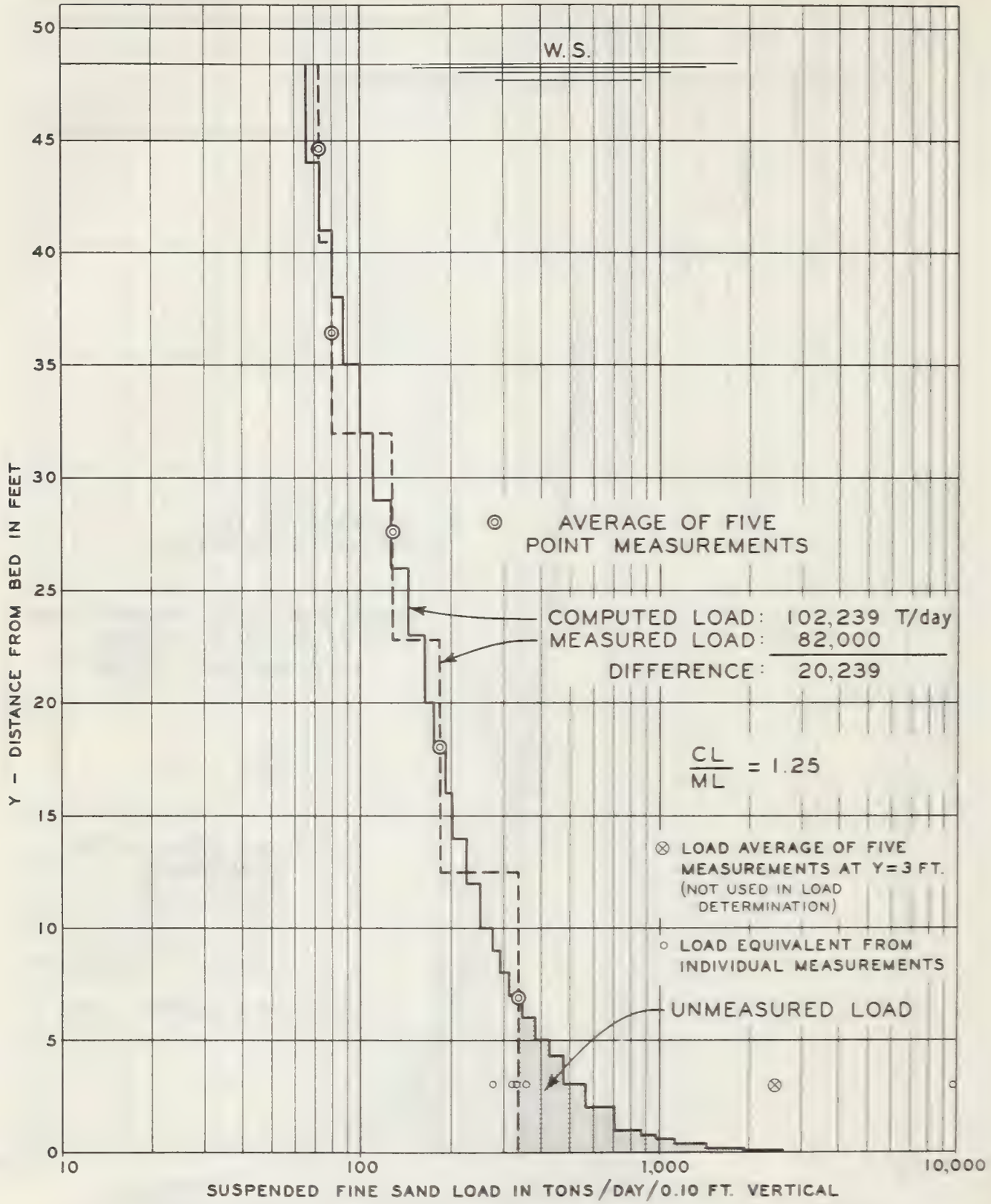
FIGURE 6. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, May 24, 1961.



PERTINENT DATA : MAY 18, 1961

Q	: 416,000 CFS	S	: 4.45×10^{-5}
WIDTH	: 1541 FT	D_{65}	: 1.34×10^{-3} FT
$\bar{u}$	: 5.65 FPS	WATER TEMP.	: 69 °F
d	: 47.7 FT	FINE SAND PORTION OF BED MATERIAL	: 39.8 %

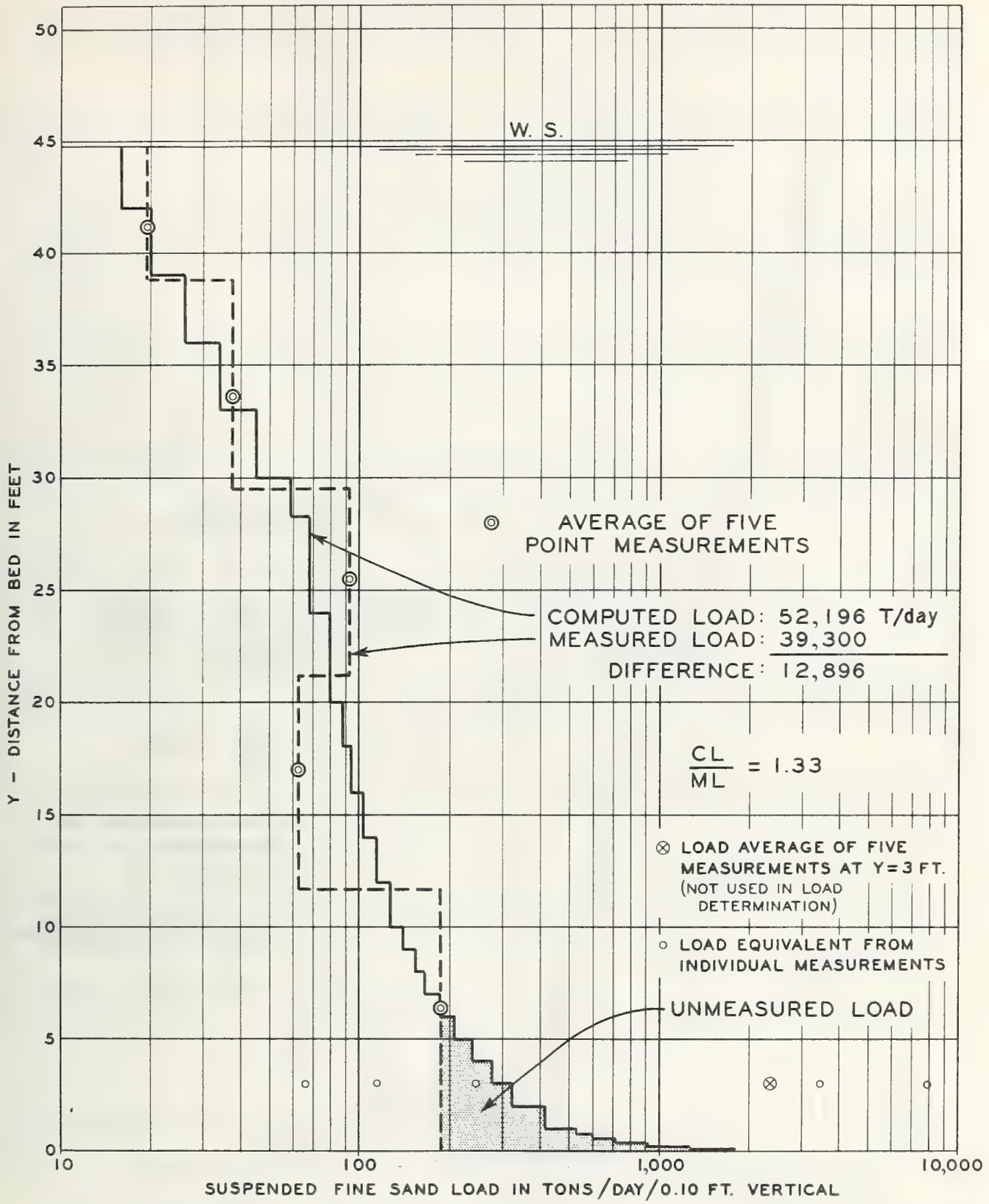
FIGURE 7.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, May 18, 1961.



PERTINENT DATA : APRIL 21, 1961

Q	: 411,000 CFS	S	: 4.45×10^{-5}
WIDTH	: 1538 FT	D_{65}	: 1.13×10^{-3} FT
$\bar{u}$	: 5.52 FPS	WATER TEMP.	: 61 °F
d	: 48.4 FT	FINE SAND PORTION OF BED MATERIAL	: 41.6 %

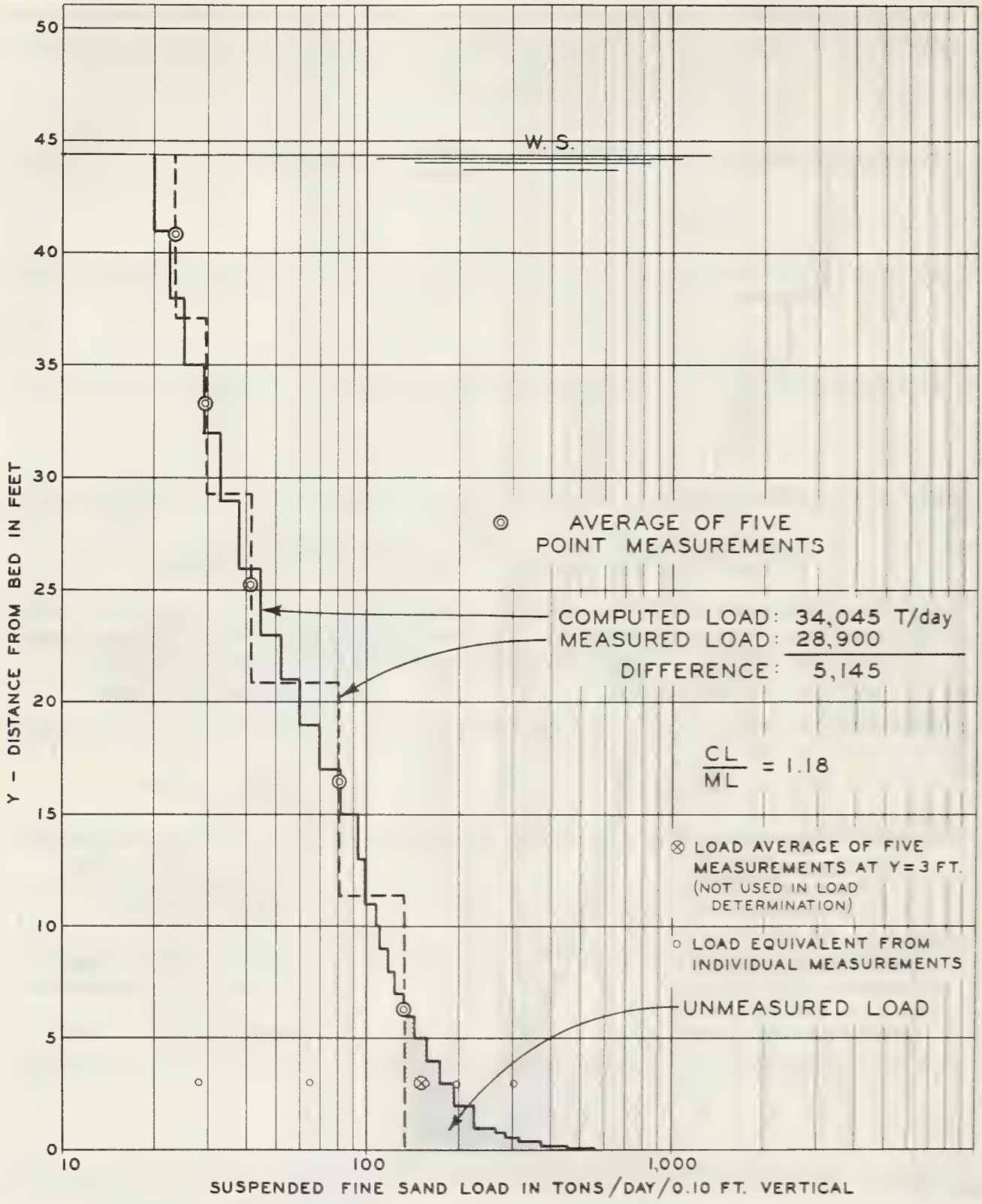
FIGURE 8. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, April 21, 1961.



PERTINENT DATA: JUNE 23, 1961

Q	284,000 CFS	S	3.74×10^{-5}
WIDTH	1333 FT	D_{65}	7.35×10^{-4} FT
$\bar{u}$	4.77 FPS	WATER TEMP.	78 °F
d	44.7 FT	FINE SAND PORTION OF BED MATERIAL	68.3 %

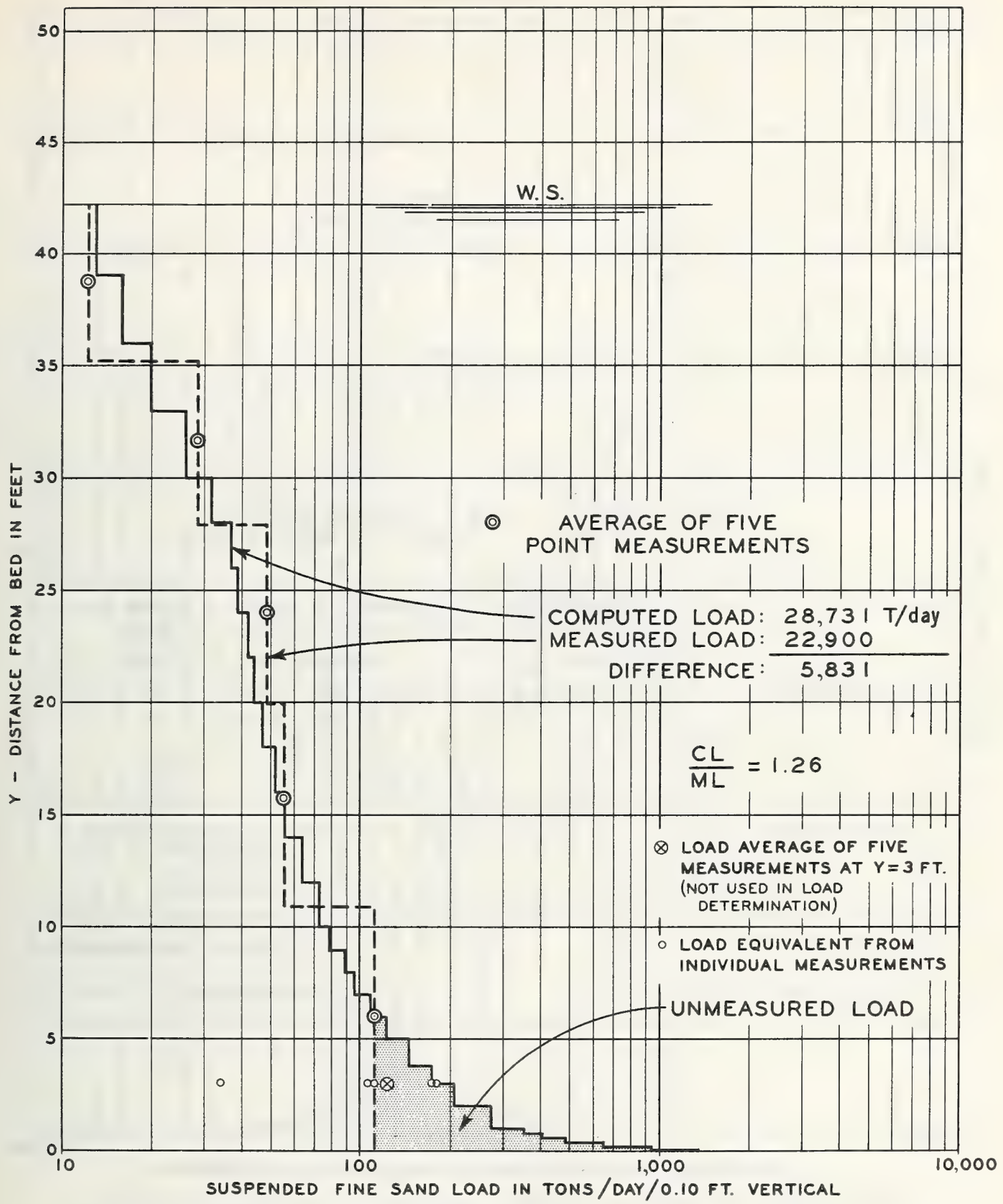
FIGURE 9.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, June 23, 1961.



PERTINENT DATA : NOV. 30, 1961

Q	: 254,000 CFS	S	: 3.74×10^{-5}
WIDTH	: 1312 FT	D_{65}	: 7.60×10^{-4} FT
$\bar{u}$	: 4.36 FPS	WATER TEMP.	: 52° F
d	: 44.4 FT	FINE SAND PORTION OF BED MATERIAL	: — %

FIGURE 10. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, November 30, 1961.



PERTINENT DATA: JULY 14, 1960

Q	233,000 CFS	S	3.42×10^{-5}
WIDTH	1290 FT	D_{65}	7.57×10^{-4} FT
$\bar{u}$	4.28 FPS	WATER TEMP.	84 °F
d	42.2 FT	FINE SAND PORTION OF BED MATERIAL	40.5 %

FIGURE 11.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, July 14, 1960.

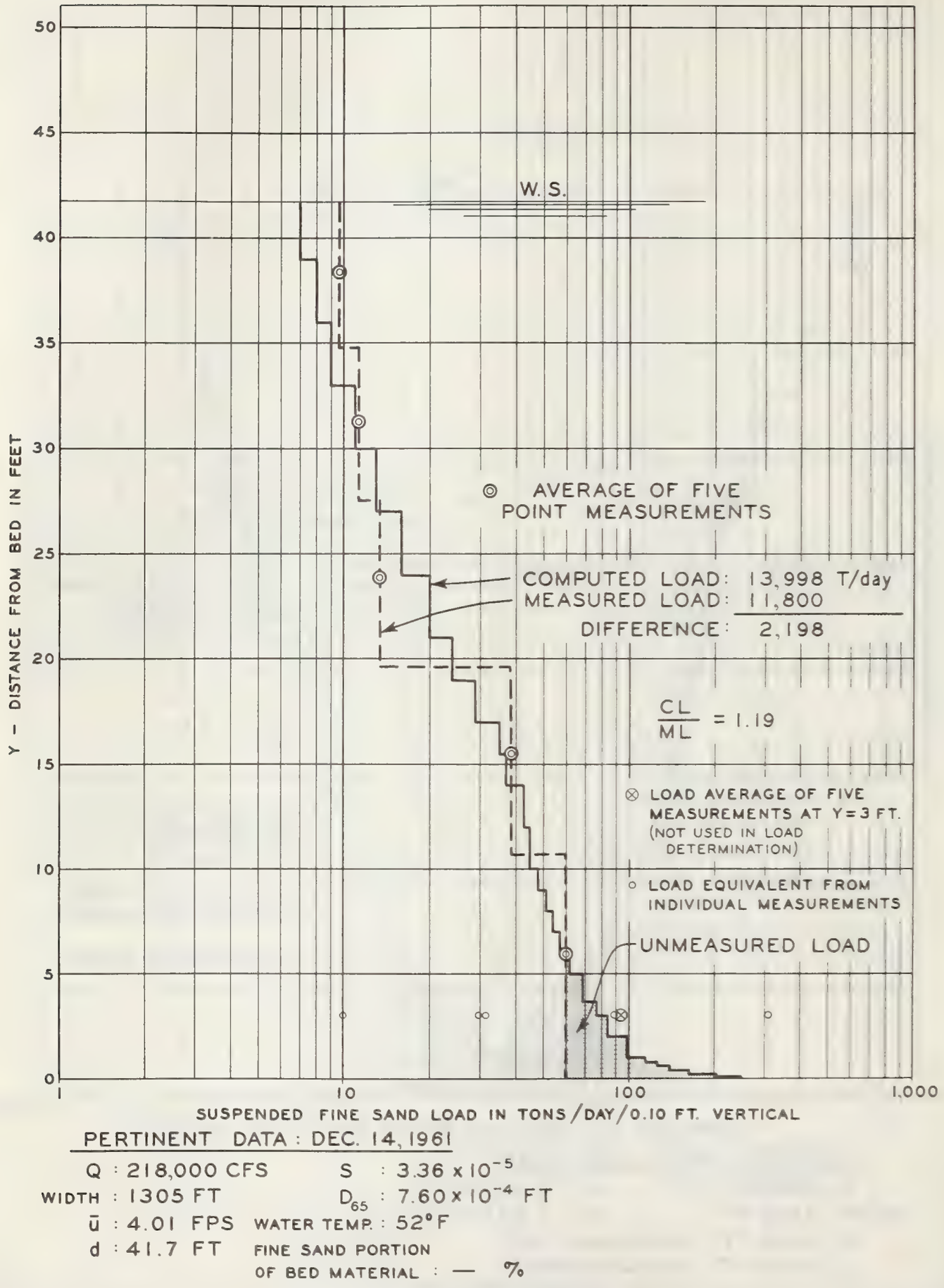
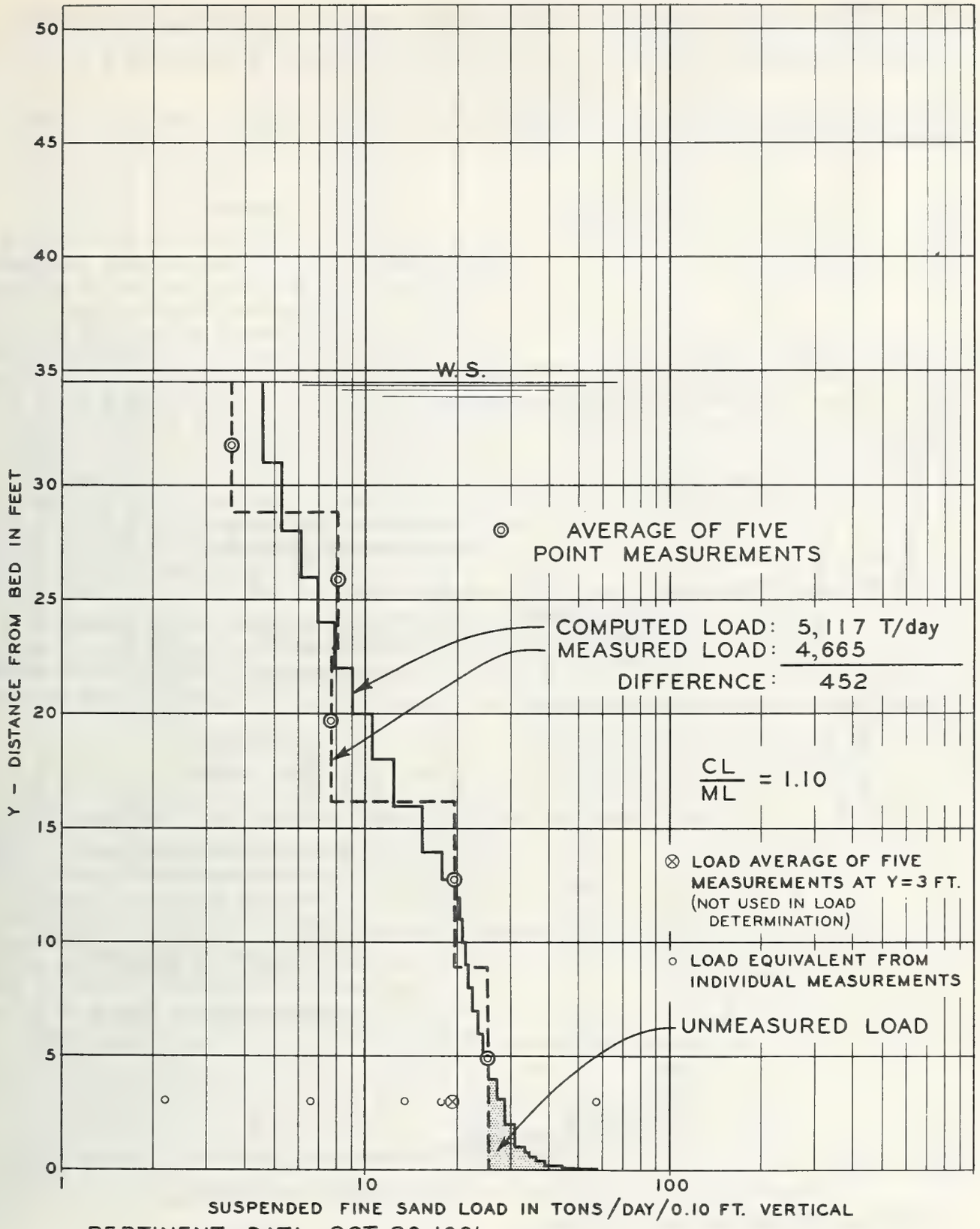


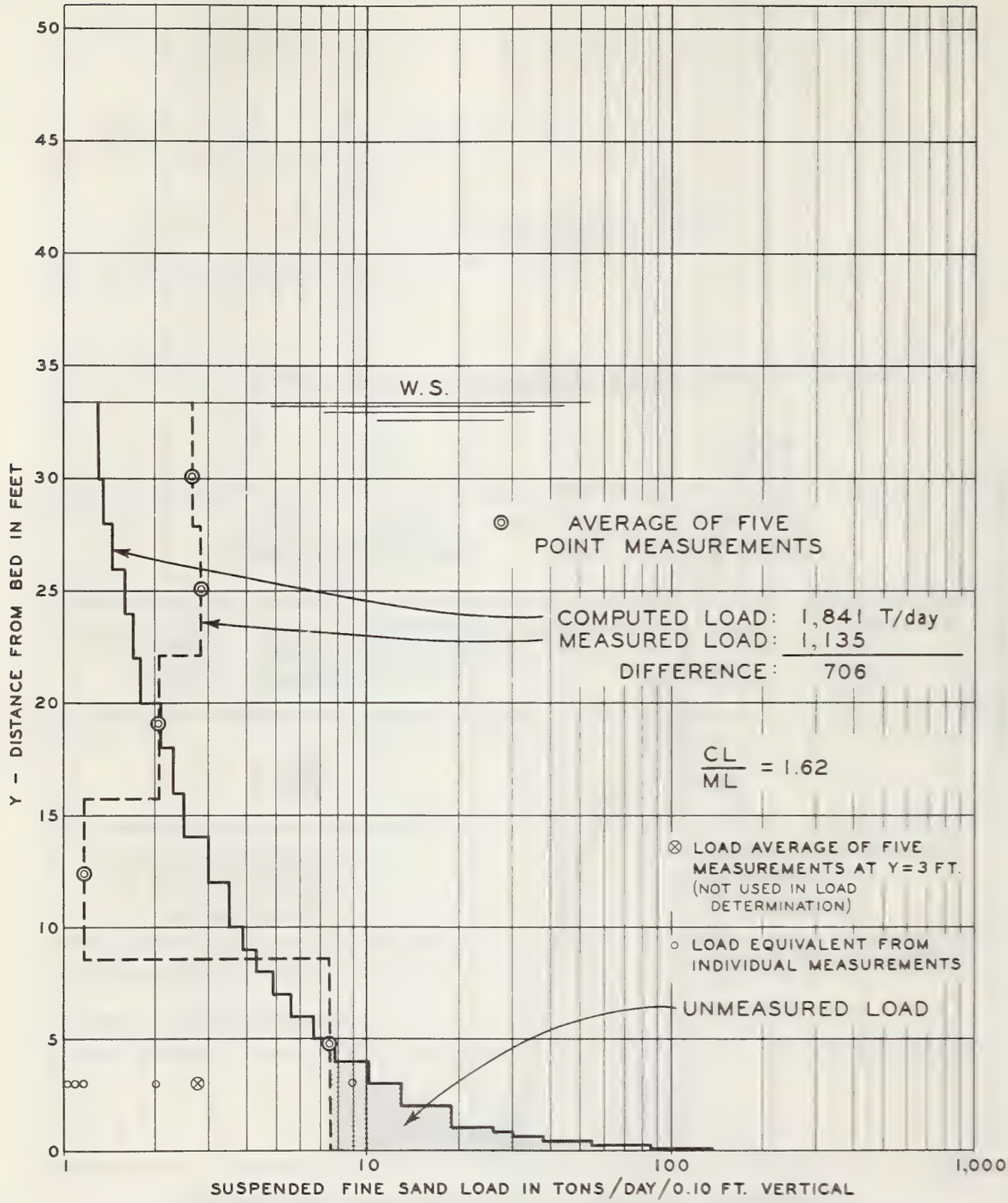
FIGURE 12. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, December 14, 1961.


Y - DISTANCE FROM BED IN FEET
W.S.
AVERAGE OF FIVE POINT MEASUREMENTS
COMPUTED LOAD: 5,117 T/day
MEASURED LOAD: 4,665
DIFFERENCE: 452
CL/ML = 1.10
LOAD AVERAGE OF FIVE MEASUREMENTS AT Y=3 FT. (NOT USED IN LOAD DETERMINATION)
LOAD EQUIVALENT FROM INDIVIDUAL MEASUREMENTS
UNMEASURED LOAD
SUSPENDED FINE SAND LOAD IN TONS/DAY/0.10 FT. VERTICAL

PERTINENT DATA : OCT. 26, 1961

Q	: 120,000 CFS	S	: 2.91×10^{-5}
WIDTH	: 1224 FT	D_{65}	: 1.00×10^{-3} FT
$\bar{u}$	: 2.84 FPS	WATER TEMP.	: 66°F
d	: 34.5 FT	FINE SAND PORTION OF BED MATERIAL	: 20.8 %

FIGURE 13. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, October 26, 1961.



PERTINENT DATA : AUG. 11, 1960

Q	: 80,800 CFS	S	: 1.96×10^{-5}
WIDTH	: 1076 FT	D_{65}	: 7.96×10^{-4} FT
$\bar{u}$	: 2.25 FPS	WATER TEMP.	: 87°F
d	: 33.4 FT	FINE SAND PORTION OF BED MATERIAL	: 54.9 %

FIGURE 14. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, August 11, 1960.

CALCULATION OF SUSPENDED LOAD DISCHARGE FROM VELOCITY AND CONCENTRATION PARAMETERS

[Paper No. 29]

By NORMAN H. BROOKS, *professor of Civil Engineering, W. M. Keck Laboratory of Hydraulics and Water Resources, California Institute of Technology;* and *visiting professor (1962-1963), Hydrodynamics Laboratory, Massachusetts Institute of Technology*

Abstract

A method is presented for the integration of suspended load discharge quickly and easily from measured or predicted distributions of velocity and sediment concentration. The procedure is based on the assumptions that (1) the velocity distribution is logarithmic, and (2) the concentration follows the suspended load equation. Each of these two functions is represented by two parameters, essentially an intercept and a slope. A further parameter is the lower limit of integration.

In applying the formulas and graphs, the investigator has complete flexibility of choice of all five parameters, because the integration procedure is not interlocked with any other assumptions.

The effect of the lower limit of integration is discussed, and a logical procedure is recommended for its selection. Unmeasured suspended load may also be easily estimated.

Introduction

The integration of the suspended load discharge of a stream is basically a mathematical problem. In the technical literature (*3,4,5,6,7*), these simple mathematical operations have often been clouded with various assumptions and assumed constants, which are not pertinent to the integration but rather are part of a more comprehensive analysis (such as the so-called "Modified Einstein Procedure" for estimating total sediment transport from measured suspended sediment samples). Since the integration depends only on the concentration and velocity distributions over the depth, it is most convenient to express the result just in parameters defining these distributions. It is the purpose of this paper to describe only this integration procedure, and thus to allow the reader flexibility to use any measurements or assumptions he desires to define these profiles.

Analysis[1]

The total suspended load discharge per unit width, q_s, may be expressed for a two-dimensional channel as the integral:

$$q_s = \int_{y_o}^{d} cu\, dy \tag{1}$$

where

$c = c(y)$ = concentration of suspended sediment (either in volume or weight per unit volume);

$u = u(y)$ = stream velocity (=downstream velocity of sediment particles);

y = distance above the bed;

y_o = lower limit of integration; and

d = total depth.

It is necessary to use a small but finite lower limit y_o instead of 0 because the sediment very close to the bed is transported as bedload, for which the usual equations for $c(y)$ and $u(y)$ do not apply, and the sediment grains no longer move at the same speed as the water.

The velocity distribution for two-dimensional flow may be expressed by the usual logarithmic velocity defect law in the form

$$u = u_{\max} + \frac{u_*}{k} \ln \frac{y}{d} \tag{2}$$

where

$u_{\max} = u(d)$ = velocity at the surface ($y=d$),

$u_* = \sqrt{\tau_o/\rho}$ = shear velocity, and

k = von Karman's constant.

Integrating over the depth to determine the average velocity, $\bar{u}$,

$$\bar{u} = \frac{1}{d}\int_0^d \left(u_{\max} + \frac{u_*}{k} \ln \frac{y}{d}\right) dy = u_{\max} - \frac{u_*}{k} \tag{3}$$

If equation 2 is rewritten with $\bar{u}$ instead of $u_{\max}$,

$$u = \bar{u} + \frac{u_*}{k}\left(1 + \ln \frac{y}{d}\right), \tag{4}$$

or if logarithms to base 10 are used,

$$u = \bar{u} + \frac{u_*}{k} + \frac{2.30u_*}{k} \log_{10} \frac{y}{d} \tag{5}$$

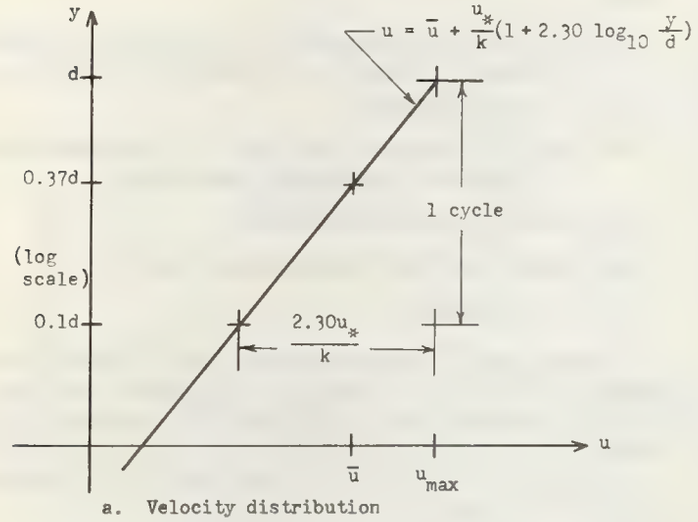
It may be noted that u_* is related to $\bar{u}$ and the friction factor (f) by the Darcy-Weisbach

[1] Most of the analyses and figure 2 were included in the author's Ph.D. dissertation. The helpful guidance and counsel of Vito A. Vanoni, his thesis adviser, is most gratefully acknowledged.

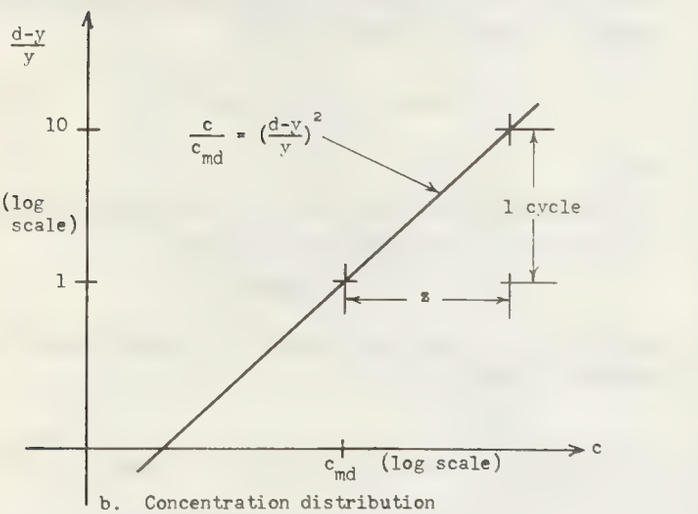
formula as applied to open channels,

$$u_*=\bar{u}\sqrt{f/8} \tag{6}$$

If the velocity distribution is plotted on semilogarithmic graph paper, it is a straight line for which $\bar{u}$ is the intercept at $y=d/e=0.37d$, and $2.30u_*/k$ is the slope (units of velocity per cycle of y) as shown in figure 1,*a*. Any logarithmic velocity distribution may thus be completely described by these two parameters ($\bar{u}$ and u_*/k).



a. Velocity distribution



b. Concentration distribution

FIGURE 1. — Definition of intercepts and slopes: *a*, Velocity distribution; *b*, concentration distribution.

The concentration distribution, $c(y)$, for each sediment size fraction can usually be represented adequately by the suspended load equation (*10,11*)[2]

$$\frac{c}{c_a}=\left(\frac{a}{d-a}\quad\frac{d-y}{y}\right)^z, \tag{7}$$

where

$c_a=c(a)$ = concentration at reference level $y=a$, and

z = exponent.

Take the reference level, $a=d/2$, and equation 7 simplifies to

$$c=c_{md}\left(\frac{d-y}{y}\right)^z, \tag{8}$$

where

$c_{md}=c(d/2)$ = middepth concentration.

If c is plotted versus $(d-y)/y$ on logarithmic graph paper, equation 8 is a straight line for which c_{md} is the intercept at $(d-y)/y=1$, and z is the slope of the straight line in cycles of c per cycle of $(d-y)/y$, as shown in figure 1,*b*. These two parameters alone are sufficient to define the distribution.

The exponent z in equation 7 or 8 has been shown theoretically by Ismail (*8*) (after nearly similar expressions given earlier by others) to be

$$z=\frac{w}{\beta k u_*}, \tag{9}$$

where

w = settling velocity of the grains, and

$\beta=\epsilon_s/\epsilon_m$ = ratio of eddy diffusion coefficient for sediment to the eddy viscosity (≈ 1).

For a limited group of experiments with flat beds of 0.10 and 0.16 mm. sand, the writer[3] has found the value of β to be very close to 1.0 (0.93 to 1.08); however, in other laboratory investigations Vanoni (*11*) reports some larger deviations of observed exponents from the theoretical, based on $\beta=1$. In the field the failure to obtain good correlation between theoretical and measured z-values, such as reported by Colby and Hembree (*4*) and Hubbell and Matejka (*7*), is probably due to difficulties of accurate determination of the various factors required to compute the z-value by equation 9.

If equations 4 and 8 are substituted into equation 1, the suspended load discharge is

$$q_s=c_{md}\int_{y_o}^{d}\left(\frac{d-y}{y}\right)^z\left[\bar{u}+\frac{u_*}{k}\left(1+\ln\frac{y}{d}\right)\right]dy \tag{10}$$

It is convenient to simplify equation 10 by the

[2] U.S. CORPS OF ENGINEERS. SEDIMENT TRANSPORTATION CHARACTERISTICS STUDY OF MISSOURI RIVER AT OMAHA, NEBRASKA. 1951. Unpublished reports.

[3] BROOKS, N. H. LABORATORY STUDIES OF THE MECHANICS OF STREAMS FLOWING OVER A MOVABLE BED OF FINE SAND. Ph.D. dissertation. On file, library of California Institute of Technology, Pasadena. 1954. (See pp. 188–194.)

change of variable $\eta = y/d$, giving:

$$q_s = c_{md}\,\bar{u}d \int_{\eta_o}^{1} \left(\frac{1-\eta}{\eta}\right)^z \left[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)\right] d\eta, \quad (11)$$

where $\eta_o = y_o/d$ is now the lower limit. By writing $\bar{u}d = q$, the discharge per unit width, and separating into two integrals, equation 11 can be rewritten as:

$$\frac{q_s}{q\,c_{md}} = \left(1+\frac{u_*}{k\bar{u}}\right) \int_{\eta_o}^{1} \left(\frac{1-\eta}{\eta}\right)^z d\eta + \frac{u_*}{k\bar{u}} \int_{\eta_o}^{1} \left(\frac{1-\eta}{\eta}\right)^z \ln\eta\, d\eta \quad (12)$$

Einstein (*6*) discussed integration of suspended load, but his formulation was not as versatile as equation 12 above. Nevertheless the same integrals are involved and Einstein published tables of computed numerical values of them, which he has designated as

$$J_1(z, \eta_o) = \int_{\eta_o}^{1} \left(\frac{1-\eta}{\eta}\right)^z d\eta, \quad (13)$$

and

$$J_2(z, \eta_o) = -\int_{\eta_o}^{1} \left(\frac{1-\eta}{\eta}\right)^z \ln\eta\, d\eta \quad (14)$$

Convenient graphs giving the values of the integrals, J_1 and J_2, have also been published (*4,7*). Equation 12 may now be conveniently rewritten in terms of a transport function, T, as

$$\frac{q_s}{q\,c_{md}} = T\left(k\frac{\bar{u}}{u_*}, z, \eta_o\right), \quad (15a)$$

where T is defined by

$$T\left(k\frac{\bar{u}}{u_*}, z, \eta_o\right) = J_1(z, \eta_o) + \left[J_1(z, \eta_o) - J_2(z, \eta_o)\right]\frac{u_*}{k\bar{u}} \quad (15b)$$

The left-hand member of equation 15a can be expressed in other ways. If the suspended load discharge concentration, q_s/q, is denoted by $\bar{c}$, then

$$\frac{q_s}{q\,c_{md}} = \frac{\bar{c}}{c_{md}} \quad (16)$$

Furthermore, if it is preferred to use the intercept c_a instead of c_{md} as the reference concentration, then equation 16 becomes

$$\frac{q_s}{q\,c_{md}} = \frac{\bar{c}}{c_a}\left(\frac{d-a}{a}\right)^z \quad (17)$$

The right-hand member of equation 15b contains only the three parameters, $k(\bar{u}/u_*)$, z, and η_o, as indicated by functional notation of T. The first two may easily be derived from measured data as explained above (see figure 1,*b*). For instance,

$$k\frac{\bar{u}}{u_*} = \frac{\bar{u}}{u_*/k} = \frac{2.30 \cdot (\text{intercept at } y = 0.37d)}{\text{slope of velocity profile graph}} \quad (18)$$

Note that it is not necessary to know u_* and k separately if a velocity profile is available. As an alternative, one may use equation 6 and obtain

$$k\frac{\bar{u}}{u_*} = k\sqrt{8/f} \quad (19)$$

The exponent z is measured as the slope of the concentration graph, or computed (less reliably) from equation 9. The only further information needed is a value for η_o, the lower limit of integration. The selection of η_o is the subject of the following section.

Lane and Kalinske (*9*) have also given a solution for suspended load integration, but it is based on their exponential concentration distribution formula, derived by assuming a constant eddy diffusion coefficient over the depth. The writer [4] has also generalized their analysis in a manner similar to the above development, but it is not included here because the suspended load equation (equation 8) is preferred.

Choice of Lower Limit of Integration

The integrals J_1 and J_2 are convergent as $\eta_o \to 0$ only for $z < 1$. Therefore, for values of z near 1 or larger than 1, the integrals change significantly with the choice of η_o. Not only is the divergence a problem, but physically the profile equations (equations 4 and 8) may not be accurate anyway for very small values of η. Thus, the limiting factor in the validity of the integration procedure is probably the accuracy of the equations representing velocity and concentration very close to the bed. Up to the present time no good way has been found to measure these profiles in the dense layer very close to the bed.

Although there are a number of possible ways to choose the lower limit $\eta_o = y_o/d$, only three will be discussed here. Since the particles can scarcely be suspended closer to the bed than two particle diameters ($2D$), Einstein (*6*) has suggested using $\eta_o = 2D/d$. For cases where the streambed is flat, this is probably as reasonable as any assumption. On the other hand for dune-covered beds for which the individual grain size is far less than the equivalent Nikuradse sand

[4] See footnote 3.

roughness for the surface, the extrapolated velocity profile will become negative near the lower limit suggested by Einstein. Thus, if $\eta_o = 2D/d$ is used, the suspended load integral may actually include some negative load contributions (cu) very close to the bed.

Another possibility for the choice of η_o is the level for which $u=0$, according to the logarithmic velocity profile. In this case from equation 4,

$$0=\bar{u}+\frac{u_*}{k}(1+\ln \eta_o).$$

Solving for η_o,

$$\eta_o = e^{-k\frac{\bar{u}}{u_*}-1}$$

Admittedly, the profile equation is not intended to be applicable down to $u=0$, but stopping the integration there certainly prevents possible unintentional negative contributions to the integral using $\eta_o=2D/d$. In fact, for the η_o giving $u=0$, the value of the transport integral $\int_{\eta_o}^{1} cu\, d\eta$ is a maximum; all other choices of η_o will mathematically result in smaller values of this integral as determined by equation 15.

A third possibility for η_o is to let it be the level at which the concentration, by extrapolation of the fitted suspended load equation, becomes equal to the concentration of the bed or of a size fraction in the bed, c_b. For this case, equation 8 gives

$$c_b = c_{md}\left(\frac{1-\eta_o}{\eta_o}\right)^z$$

and since $\eta_o << 1$,

$$\eta_o = \left(\frac{c_{md}}{c_b}\right)^{\frac{1}{z}}$$

It would certainly be physically unreasonable to use a smaller lower limit than this, as is sometimes the case when $\eta_o=2D/d$ or $u(\eta_o)=0$ is used, for then the integration would include values of $c > c_b$.

The three possibilities outlined above may now be summarized as follows:

(a) $\eta_o = \frac{2D}{d}$ (Einstein bed layer) (20)

(b) $\eta_o = e^{-k\frac{\bar{u}}{u_*}-1}$ $u(\eta_o)=0$ (21)

(c) $\eta_o = \left(\frac{c_{md}}{c_b}\right)^{\frac{1}{z}}$ $c(\eta_o)=c_b$ (22)

Perhaps the most reasonable choice of the three is the *largest* η_o, because it has been shown that it is physically unreasonable to go lower than any one of the three. Further experimental investigation is needed to verify whether this is a satisfactory approach, at least for approximate solutions.

Because of its simplicity, choice (b) warrants special attention. As equation 15 stands, $q_s/q\ c_{md}$ is a function of the three parameters $k\frac{\bar{u}}{u_*}$, z, and η_o; but since η_o is a function of $k\frac{\bar{u}}{u_*}$ by equation 21, η_o may be eliminated as an explicit parameter. The dimensionless transport given by equation 15 depends then only on z and $k\frac{\bar{u}}{u_*}$ and may be represented on a single chart.

Figure 2 was constructed by the writer[5] on this basis from the values of the integrals given by Einstein (*6*).

For a given z in figure 2, the ratio $q_s/q\ c_{md}$ increases when $k\frac{\bar{u}}{u_*}$ (or $k\sqrt{8/f}$) increases. This relation is partly the result of changing the shape of the velocity profile and partly the effect of reducing the lower limit η_o. The latter is especially important for the larger z values, for which the concentration profile is very steep and most of the suspended load moves near the bed; in this case, a small change in the η_o value can change the value of the q_s integral a large amount. In fact, since the value of η_o based on $u(\eta_o)=0$ may not be the best value anyway, considerable error can be expected when $q_s/q\ c_{md}$ gets very large on the chart. Consequently, the curves have not been extended for $q_s/q\ c_{md} > 50$, or for $z > 2$.

It may be observed from figure 2 that for $z < 0.3$, $q_s/q\ c_{md}$ may be considered equal to 1.0 for all practical purposes. The slight drop below 1.0 for the curves for the smaller values of $k\frac{\bar{u}}{u_*}$ at small z is real and may be explained as follows. Although concentrations are larger toward the bed, velocities are greater toward the surface. Usually the nonuniformity of concentration is much more pronounced than for the velocity, resulting in a weighted average concentration $\bar{c}$ greater than the middepth concentration c_{md}. However, the situation is reversed for *small* z (relatively uniform c) and small $k\frac{\bar{u}}{u_*}$ (high roughness, steep velocity profile), and the average $\bar{c}$ does actually become less than c_{md}.

Application of Results

Example of Use of Figure 2

The use of figure 2 may be illustrated by the

[5] See footnote 3.

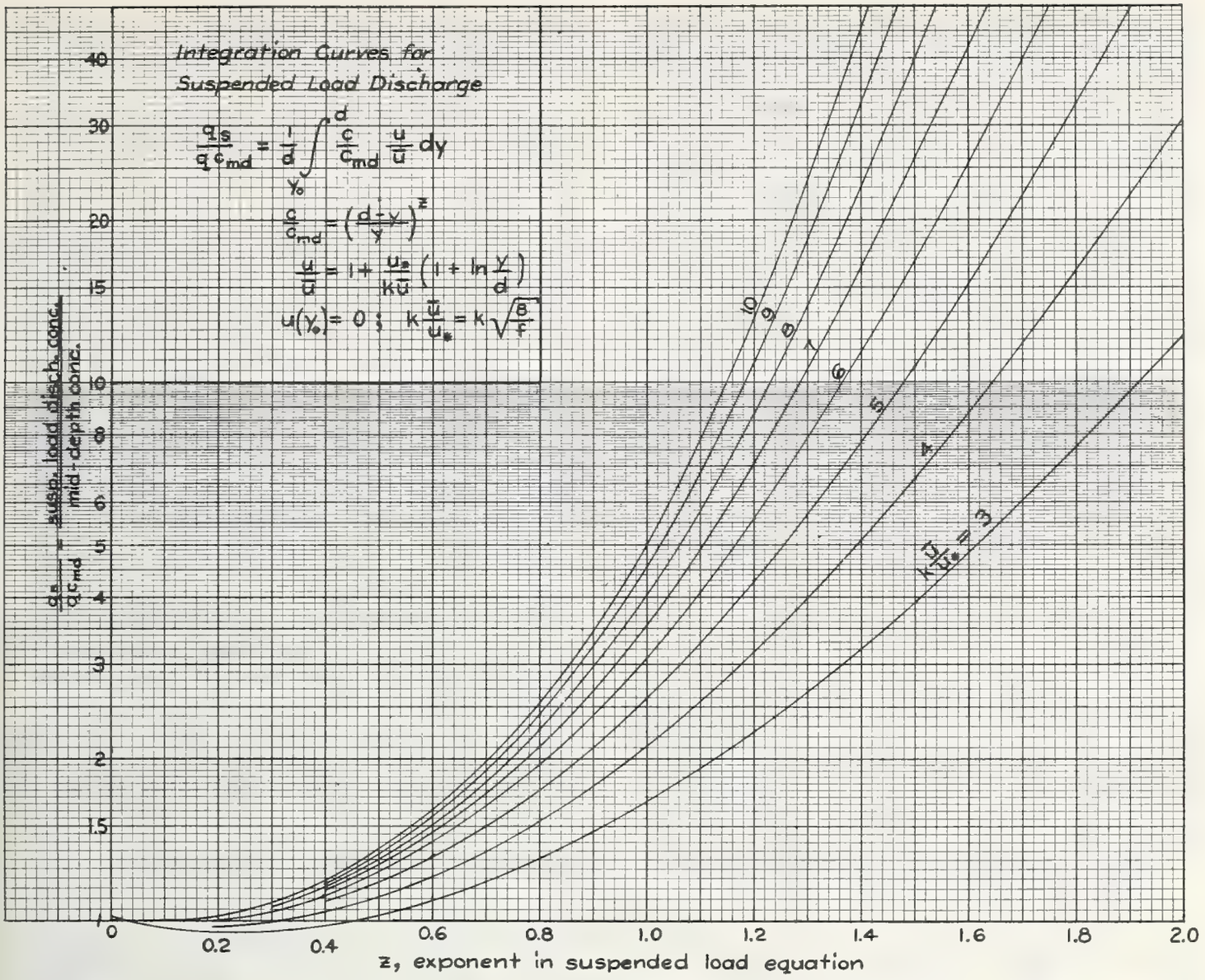

Integration Curves for
Suspended Load Discharge
$\frac{q_s}{q\,c_{md}} = \frac{1}{d}\int_{y_0}^{d} \frac{c}{c_{md}}\frac{u}{\bar{u}}\,dy$
$\frac{c}{c_{md}} = \left(\frac{d-y}{y}\right)^z$
$\frac{u}{\bar{u}} = 1 + \frac{u_*}{k\bar{u}}\left(1 + \ln\frac{y}{d}\right)$
$u(y_0) = 0\ ;\ k\frac{\bar{u}}{u_*} = k\sqrt{\frac{8}{f}}$
$\frac{q_s}{q\,c_{md}}$ = susp. load disch. conc. / mid-depth conc.
$k\frac{\bar{u}}{u_*} = 3$
z, exponent in suspended load equation

FIGURE 2. — Integration curves for suspended load discharge by equations 15 and 21.

following example. Suppose that from a measured velocity profile for a wide stream 3 ft. deep it is found that $\bar{u}=u(0.37d)=3.5$ ft./sec. The slope of the velocity profile=1.25 ft./sec. per cycle=$\frac{2.3u_*}{k}$; therefore $\frac{u_*}{k}=0.54$. In addition, when c is plotted on logarithmic paper vs. $(d\text{-}y)/y$, the slope of the line of best fit is $z=1.1$, and the line intersects $(d\text{-}y)/y=1$ at $c_{md}=0.05$ g./l.=50 p.p.m. Therefore,

$$k\frac{\bar{u}}{u_*} = \frac{3.5}{0.54} = 6.5,$$

$$z = 1.1$$

By figure 2,

$$\frac{q_s}{q\,c_{md}} = 4.5,$$

and hence,

$$\frac{q_s}{q} = 4.5\,c_{md} = 4.5\,(50) = 225 \text{ p.p.m.}$$

Since the discharge per unit width is

$$q = \bar{u}\,d = (3.5)\ (3) = 10.5 \text{ c.f.s./ft.},$$

the suspended load discharge per unit width is

$$q_s = \frac{225}{10^6} \times 10.5 \times 62.4 = 0.15 \text{ lb./ft.-sec.}$$

Laboratory Experiments

The writer has applied figure 2 to laboratory experiments in a 10.5-inch wide flume, and compared the results with measured total sediment loads. Two different bed sands were used, with mean sieve diameters of 0.145 mm. and 0.088 mm., respectively. Only flat bed cases were used because no velocity and concentration profiles were measured for the dune bed cases.

A detailed account of the experiments, including velocity and concentration profiles, is given

in the writer's Ph. D. thesis,[6] and a briefer description including tabulation of the basic data (but not the profiles) is available in reference (*2*).

A rigorous comparison of experimental results with the calculated suspended load discharge is not possible, because the measured sediment discharge concentration, $\bar{C}$, includes also the bedload and because the centerline profiles of concentration and velocity on which the integration is based are not truly representative of the whole cross section. Nevertheless, with the high transportation rates associated with the runs for which profiles were obtained, the bedload is known to be small. Furthermore, it has been observed that the concentration does not vary substantially going across the channel at any given level. Since most of the load is carried close to the bed anyway, the distortion of the velocity distribution by the walls at the upper levels does not introduce a very large error. Consequently, the order of magnitude of the measured concentration, $\bar{C}$, should be the same as the calculated ratio of suspended load discharge to water discharge, q_s/q.

When any of the integration procedures are used, it is important to have the suspended load equation fit the observed concentrations well only where the concentrations are large. If the curve deviates moderate amounts when the concentrations are relatively small, as was often the case in the plotted concentration profiles, the error introduced in the total transportation rate is still small.

The basic experimental data required for the calculation of the integrals are listed in table 1. As indicated, the parameter $k\frac{\bar{u}}{u_*}$ was determined from the intercept and slope of velocity distribution (on the centerline of the flume). The concentration distribution yields c_{md} (intercept) and z (slope). The mean discharge concentration was measured in a vertical part of the return flow circuit at a point where all the load was suspended.

The three values of η_o, which are listed in table 1, were calculated by equations 20 to 22 and represent various possibilities for the lower limit of integration. The values of the dimensionless ratio, $q_s/q\ c_{md}$, listed in table 1, were calculated from $k\frac{\bar{u}}{u_*}$, z, and η_o by equation 15 and appropriate charts in references (*4*), (*6*), or (*7*), and may be compared with the measured ratio C/c_{md}. (For column 2, the values of $q_s/q\ c_{md}$ were obtained directly from figure 2, except for run 7.)

From an inspection of the table, it is clear that the agreement between the values is not perfect, but qualitatively quite good. When the

[6] See footnote 3.

TABLE 1.—*Integration for suspended sediment discharge for flume experiments* [1]

BASIC DATA

Run No.	Mean sieve diameter of bed (D)	Mean depth (d)	Velocity distribution			Concentration dist.		Measured mean discharge concentration ($\bar{C}$)
			Mean = u (0.37d) ($\bar{u}$)	Slope $\left(2.30\frac{u_*}{k}\right)$	$k\frac{\bar{u}}{u^*}$	Mid-depth conc. (c_{md})	Slope (z)	
	Mm.	*Feet*	*Ft./sec.*	*Ft./sec.*		*G./l.*		*G./l.*
3	0.145	0.243	2.32	1.07	4.98	0.20	1.36	1.95
4	.145	.236	2.43	1.05	5.32	.54	1.20	2.45
6	.145	.195	2.35	.99	5.45	.29	1.49	2.45
7	.145	.243	2.38	.90	6.07	.092	1.78	2.15
21	.088	.236	2.37	1.08	5.03	3.0	.76	4.85
29	.088	.280	2.36	1.07	5.05	1.9	.83	3.45

RESULTS OF INTEGRATION

Run No.	z	$k\frac{\bar{u}}{u_*}$	Values of lower limit $\eta_o=\frac{y_o}{d}$			Values of the integral, (q_s/qc_{md}), calculated for η_o as listed			$\bar{C}/c_{md}$ (measured total load)
			(1)	(2)	(3)				
			$\eta_o=\frac{2D}{d}$	$u(\eta_o)=0$	$c(\eta_o)=c_b$ [2]	(1)	(2) [3]	(3)	
3	1.36	4.98	0.0039	0.00255	0.00145	6.6	6.8	6.6	9.7
4	1.20	5.32	.0040	.0018	.0014	4.45	4.7	4.7	4.55
6	1.49	5.45	.0049	.0016	.00325	10.5	13	11.5	8.45
7	1.78	6.07	.0039	.00084	.0044	40	65	39	23.5
21	0.76	5.03	.0025	.0024	.00029	1.62	1.65	1.57	1.62
29	0.83	5.05	.00205	.00235	.00033	1.83	1.85	1.75	1.82

[1] See footnote 3.
[2] c_b=1,450 gr./l.
[3] Values read directly from figure 2, except for run 7.

value of the exponent z is small (less than 1) and hence $q_s/q\ c_{md}$ is small, the agreement of all the values is excellent; for the theoretical calculation it appears that any one of these choices of the lower limit would be satisfactory.

However, for the higher z values (greater than 1.3 or 1.4), the calculated $q_s/q\ c_{md}$ is significantly larger than the experimentally determined concentration ratio, $\bar{C}/c_{md}$. The discrepancy results from the fact that when z is large, a major part of the load travels very close to the bed where the extrapolated concentration and velocity profiles may not be accurate. None of the choices of the lower limit gives a good value of $q_s/q\ c_{md}$ when this ratio is large.

A further example of the application of figure 2 has been included as table 2 for the concentration profiles obtained for individual sieve fractions for run 29. The exponent z ranges from 0.53 to 1.27, and the values obtained from figure 2 compare favorably with the measured ratio $\bar{C}/c_{md}$ for each sieve fraction.

TABLE 2.—*Integration of suspended sediment for individual sieve fractions for run 29, when* $k\frac{\bar{u}}{u_*}=5.05$

Item	Tyler sieve fractions				Overall (data from table 1)
	150	170	200	250	
z, exponent	1.27	1.03	0.93	0.76	0.83
$q_s/q\ c_{md}$, by fig. 2	5.3	2.82	2.26	1.65	1.85
$\bar{C}/c_{md}$, measured	3.92	2.66	1.92	1.43	1.82

For all the present runs where the sand bed was smooth, the level $\eta_o=2D/d$ (table 1) is higher than the level for which $u(\eta_o)=0$, and thus it might be reasoned that the former would be a better choice. However, when the bed becomes covered with dunes, the friction factor increases and the situation is reversed. For example, assume the typical values $f_b=0.08$ and $k=0.30$; by equation 19,

$$k\frac{\bar{u}}{u_*}=k\sqrt{\frac{8}{f_b}}=3.0,$$

and by equation 21,

$$\eta_o=e^{-3.0\ -1}=0.018$$

Now η_o is considerably more than $2D/d$ for any of the runs, and the use of $\eta_o=2D/d$ in a case like this would include negative values of the integrand in the load integral. Since Barton and Lin (*1*) have shown in the laboratory that the suspended load equation can be applied reasonably well to flows over a dune-covered bed, then the use of η_o for which $u(\eta_o)=0$ (and the chart in fig. 2) is recommended in place of the lower limit $\eta_o=2D/d$ for such cases.

One of the advantages of the diagram in figure 2 is the avoidance of some of the computational error accumulated in using the charts for J_1 and J_2, and equation 15. First, in reading the charts, which are on logarithmic graph paper with 3-inch cycles in reference (*4*) or (*7*), interpolation is usually required, making it difficult to obtain better than 2-place accuracy. Then making the calculation according to equation 15 usually involves taking the difference between numbers of the same order of magnitude, increasing the error further.

Estimation of Unmeasured Suspended Load

In sampling natural streams for determination of the suspended load discharge, a depth integrating sampler is frequently used. Since it samples the flow in proportion to the velocity at each level as it slowly traverses the depth, the concentration of sediment in the integrated sample is the weighted average concentration expressed as

$$c(y_1)=\frac{\int_{y_1}^{d} cu\,dy}{\int_{y_1}^{d} u\,dy}, \tag{23}$$

where the value of $\bar{c}$ obtained is a function of y_1, the lower limit of the traverse. The numerator is given by equation 10 with y_1 replacing y_o, while the denominator can be considered the partial discharge $q(y_1)$ which flows above $y=y_1$. From equation 4, and changing to the dimensionless elevation η, the partial discharge is

$$q(y_1)=\int_{y_1}^{d} u\,dy=\bar{u}\,d\int_{\eta_1}^{1}\left[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)\right]d\eta \tag{24}$$

Integrating,

$$q(\eta_1)=\bar{u}d\left(1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1\ln\eta_1\right) \tag{25}$$

Equation 23 may be written as

$$\bar{c}(\eta_1)=\frac{q_s(\eta_1)}{q(\eta_1)}=\frac{q_s(\eta_1)/q}{q(\eta_1)/q} \tag{26}$$

and substitution of equations 15 and 25 yields

$$\frac{\bar{c}(\eta_1)}{c_{md}}=\frac{T\left(k\frac{\bar{u}}{u_*},z,\eta_1\right)}{1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1\ln\eta_1} \tag{27}$$

For the total suspended load transport with lower limit η_o, equation 15 is

$$\frac{\bar{c}(\eta_o)}{c_{md}}=T\left(k\frac{\bar{u}}{u_*},z,\eta_o\right) \tag{28}$$

Dividing equation 28 by equation 27 gives

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}=(1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1\ln\eta_1)\frac{T(k\frac{\bar{u}}{u_*},z,\eta_o)}{T(k\frac{\bar{u}}{u_*},z,\eta_1)} \quad (29)$$

If the parameters $k\frac{\bar{u}}{u_*}$, z, η_o and η_1 are known or can be estimated, then the ratio of the desired concentration $\bar{c}(\eta_o)$ (for the *total* suspended load) to measured concentration $\bar{c}(\eta_1)$ can be readily determined by equation 29, where T is defined in equation 15b.

The unmeasured suspended load transport is usually defined as

$$\text{Unmeasured } q_s=q[\bar{c}(\eta_o)-\bar{c}(\eta_1)]$$
$$=q\bar{c}(\eta_1)[\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}-1] \quad (30)$$

where q is the full discharge and the ratio is given in equation 29. Since the quantity $qc(\eta_1)$ is called the measured suspended load transport, the term in brackets in equation 30 is the ratio of unmeasured to measured suspended load discharge.

The previous example may be extended to illustrate the use of equation 29. Suppose that z is estimated to be 1.1 and that a depth integrated sample taken down to $y=0.10d$ has a mean concentration of 0.12 g./l. The mid-depth concentration c_{md} is now presumed unknown. Evaluation of equation 29 gives

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}=(1-0.10-\frac{2.30}{6.5}0.10\log_{10}0.10)$$
$$=\frac{T(6.5,1.1,\eta_o)}{T(6.5,1.1,0.10)}.$$

If figure 2 (with η_o thus implied to be $u(\eta_o)=0$, or $\eta_o=0.00055$) is used,

$$T(6.5,1.1,\eta_o)=4.65.$$

With the use of the charts for J_1 and J_2 in (*4*) or (*7*) and the definition of T in equation 15b,

$$T(6.5,1.1,0.10)=J_1(1.1,0.10)+$$
$$\frac{1}{6.5}[J_1(1.1,0.10)-J_2(1.1,0.10)]$$
$$=1.56+\frac{1}{6.5}(1.56-2.27)$$
$$=1.45$$

Then

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}=(0.935)\frac{4.65}{1.45}=3.00,$$

and finally

$$\bar{c}(\eta_o)=\frac{q_s}{q}=3.00(0.12)=0.36 \text{ g./l.}$$

In this case the unmeasured suspended load discharge would be 2.00 times the measured load discharge. Above the level $y=0.10d$, the water discharge is 93.5 percent of the whole, and the suspended sediment discharge is $\frac{1.45}{4.65}=31$ percent.

Equation 25 may also be used to explain the anomology in figure 2 where the curve for $k\frac{\bar{u}}{u_*}=3$ rises slightly above the ordinate 1.00 as $z\to 0$. For $z=0$, $c=$constant, and equation 11 reduces to equation 24, that is

$$\frac{q_s}{\bar{u}d\,c_{md}}=\int_{\eta_o}^{1}[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)]d\eta=\frac{q(\eta_o)}{\bar{u}d}$$
$$=1-\eta_o-\frac{u_*}{k\bar{u}}\eta_o\ln\eta_o \quad (31)$$

For $k\frac{\bar{u}}{u_*}=3$, equation 21 gives

$$\eta_o=e^{-3-1}=0.0183$$

and by equation 31,

$$\frac{q_s}{\bar{u}d\,c_{md}}=1.006,$$

or 0.6 percent in error. This discrepancy is due to the fact that the average velocity is based on integration from zero (equation 3) rather than the finite lower limit η_o. Since the error is so small and becomes even smaller for larger values of $k\frac{\bar{u}}{u_*}$, no adjustment was made in figure 2.

Summary

Equation 15 is recommended for integration of the suspended load transport. Numerical calculations may be made with the aid of published graphs of J_1 and J_2 (*4,7*) for any choice of η_o, the lower limit of integration. The selection of the lower limit might reasonably be selected as the largest of the three choices given by equations 20, 21, and 22. Use of equation 21 for η_o is mathematically the simplest, because the complete integration equation (equation 15) can then be represented in one chart (fig. 2). A numerical example and flume data illustrate the application of the equations and figure 2.

From the integration equation the suspended load transport measured by a depth integrating sampler can be corrected for the suspended load in the unsampled zone below the lowest point reached by the sampler during its traverse.

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EFFECT OF TEMPERATURE ON RIVERBED CONFIGURATION: ITS POSSIBLE STAGE-DISCHARGE IMPLICATIONS

[Paper No. 31]

By WALTER C. CAREY, *assistant chief, Engineering Division, U.S. Army Engineer District, New Orleans*

On the basis of field and other studies, it is concluded that the bed configuration of an alluvial river varies in accordance with water temperature; that is, crossings and bends are lowered and amplitudes of dunes or sand waves are reduced in some relationship with reduced water temperature (fig. 1).[1] These several effects substantially reduce resistance to flow, which, in turn, should produce the end result of a lowered water surface; that is, with colder water a given flow should move past any given point at a lower stage.

Collection and Analysis of Basic Data

The fathometer was the instrument used to collect the basic data; that is, continuous thalweg profiles. Vertical and longitudinal controls were good. Longitudinal control was provided by taking fixes on navigation lights and controlling boat speed by tachometer. Lateral control was somewhat less satisfactory. The longitudinal scale for the first runs was 200 feet to the inch and 300 feet to the inch for the later runs. Either scale shows the riverbed in great detail.

Seven thalweg profile runs, totaling nearly 3,000 miles, were made from New Orleans to Old River (200 miles) and return at the nominal stage of 37 feet Red River Landing gage (fig. 2).[2] River temperatures ranged from 42° to 83° F., the latter being for the August 1958 [3] run, which is the basis for comparison with all the other runs made at lower temperatures. (table 1). The temperature extremes encountered in this field work are only moderately less than the all-time records at the Eagle Street water intake of the New Orleans Sewerage and Water Board.[4] The temperatures at the intake were found to agree closely with those observed by the survey boat at various locations between New Orleans and Old River during the several thalweg profile runs. As nearly as is known, the only variable involved in these seven thalweg profile runs was temperature.

Practical Implications

Some practical implications of these conclusions are suggested by examining two unresolved hydraulic anomalies: First, there is the fact that the 1950 flood, the principal criterion for determining the "project flood flowline" for the lower Mississippi, was out of step with some other historic floods. The fact that this flood crest passed New Orleans at a temperature of 50° F. suggests the possibility of temperature as the factor that might account for this disagreement with other historic floods, the crests

[1] The chief factor in the variation in wave size in figure 1 is volume of flow, and any effect of temperature is completely obscured. Figure 1 serves here simply to show nature of waves.

[2] Another series of thalweg profile runs at a nominal stage of 12.5 feet Red River Landing gage has been completed. The analysis of these data was not completed at the time of this paper, however, and will be presented in a subsequent paper.

[3] The August 1958 rise was notable in that it was the highest stage at Red River Landing in August in 30 years. It thus provided a much needed basis (82°-83° F.) for comparison with lower temperature flows at the same stage.

[4] Maximum temperature of record was 90° F., Aug. 17, 1937; minimum temperature was 36°, Jan. 30, 1948.

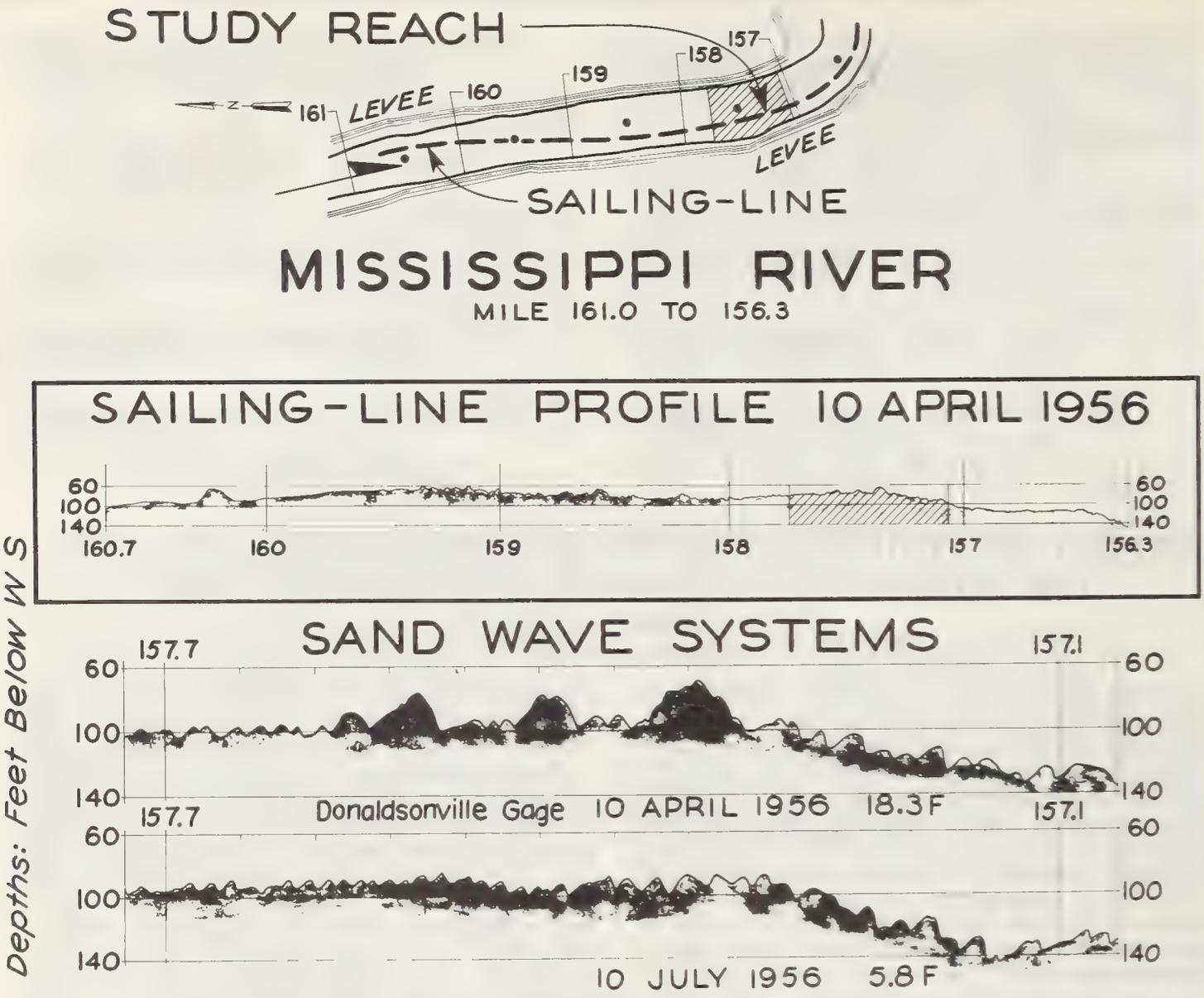

FIGURE 1. — Segments of sailing-line (thalweg) profiles at 18.3 and 5.8 feet, respectively, Donaldsonville gage, showing sand-wave systems.

of which arrived some weeks later than the 1950 crest and with somewhat higher water temperatures. Similarly, water temperature may also account for the unexplained behavior of the Mississippi at St. Louis, Mo., where it is reported that summer floods are substantially less in volume of flow as compared with spring floods reaching the same stage; that is, the river capacity seems to be greater with the colder water.

Scope of Study

This paper reports the results of field investigations concerning changes in riverbed configuration and also presents certain official published discharge records, arranged in such a manner as to provide additional support for the thesis of this paper. The full hydraulic implications of this empirical study are left for fuller exposition as a result of some future broad scale investigation and study of this aspect of river flow.

The riverbed statistics (tables 2 to 8), which provide the principal basis for this paper and its conclusions, were derived from measurements and studies made on the thalweg profiles resulting from the seven field runs over the period August 1958 to February 1962. Although temperature is suggested as being the only known variable, the riverbed statistics do not seem to conform, or fall into order, to the degree that might be expected, if temperature were the sole influence. There is a possibility that some element is involved, such as a "lag" in riverbed configuration adjustment, necessitated as a result of flow or temperature changes. In a model, time could be allowed for such adjustments, but, on the natural river, the field obser-

TABLE 1.—*Mississippi River water temperature (° F.), 1958–62*

Days of month	June 1958	July 1958	Aug. 1958	Aug. 1958 [1]	Dec. 1958	Jan. 1959	Feb. 1959	Feb. 1959[1]	Feb. 1960	Mar. 1960	Apr. 1960	Apr. 1960 [1]	Jan. 1961	Feb. 1961	Mar. 1961	Mar. 1961 [1]	Apr. 1961	May 1961	May 1961[1]	June 1961	June 1961[1]	Dec. 1961	Jan. 1962	Feb. 1962	Feb. 1962 [1]
1	.....	80	84	.....	54	.....	.....	.....	49	.....	.....	.....	43	44	53	.....	56	61	.....	70	70	53	45	40	.....
2	77	80	.....	.....	54	44	45	.....	48	42	.....	.....	43	43	51	.....	58	62	.....	70	70	53	44	41	.....
3	77	80	.....	.....	53	.....	45	.....	48	42	.....	.....	43	43	51	.....	58	63	.....	70	70	53	44	42	.....
4	77	.....	84	83	53	.....	45	.....	48	42	50	.....	43	40	53	.....	58	63	.....	70	70	52	44	42	.....
5	78	.....	84	82	53	43	45	.....	49	.....	51	.....	48	41	52	.....	56	64	.....	71	71	51	44	43	.....
6	78	.....	84	82	.....	43	45	.....	.....	.....	53	.....	48	44	54	.....	56	65	.....	71	72	51	44	42	.....
7	.....	80	84	82	.....	43	.....	.....	.....	42	54	.....	48	41	54	.....	56	65	.....	72	73	50	44	43	.....
8	.....	80	84	83	53	43	.....	.....	48	42	54	.....	46	41	54	.....	56	65	66	73	.....	50	43	44	.....
9	80	81	.....	.....	53	43	46	43	48	43	.....	.....	46	40	53	.....	57	64	65	73	.....	51	43	44	.....
10	81	81	.....	.....	52	.....	.....	42	48	43	.....	.....	47	42	54	.....	56	64	64	74	.....	50	42	44	.....
11	81	81	84	82	52	.....	46	42	48	.....	55	.....	43	42	54	.....	60	65	65	74	.....	51	42	44	.....
12	81	.....	84	83	51	43	45	43	49	.....	55	.....	43	42	53	.....	60	65	65	75	.....	51	42	45	.....
13	82	.....	84	82	.....	44	46	.....	.....	42	54	.....	43	42	53	.....	58	66	.....	75	.....	52	40	45	.....
14	.....	82	84	82	.....	44	.....	.....	.....	42	54	.....	44	42	54	.....	56	66	.....	75	.....	51	40	45	.....
15	.....	82	84	83	48	44	.....	.....	48	42	.....	.....	43	44	54	.....	56	65	.....	75	...	52	40	45	.....
16	83	82	.....	.....	47	44	46	43	48	42	.....	.....	43	42	54	.....	56	66	.....	75	.....	52	40	46	47
17	83	82	.....	.....	46	.....	46	42	48	42	.....	.....	43	42	54	.....	58	66	.....	76	.....	51	40	46	46
18	84	83	84	83	46	.....	46	.....	49	.....	56	.....	43	43	58	.....	58	66	.....	76	.....	51	39	46	47
19	84	.....	83	.....	45	43	45	.....	49	.....	.....	.....	44	45	58	.....	58	66	.....	77	.....	50	39	47	47
20	83	.....	83	.....	.....	43	45	.....	.....	43	58	.....	42	46	60	56	58	68	.....	77	.....	50	39	47	.....
21	.....	84	83	.....	.....	.....	.....	.....	.....	44	58	56	43	50	60	56	57	68	.....	76	.....	50	39	47	.....
22	.....	84	83	.....	45	43	.....	.....	44	44	59	57	44	48	58	56	57	68	.....	76	.....	50	39	47	.....
23	82	84	.....	.....	45	44	46	.....	44	45	.....	59	43	53	58	58	58	68	.....	76	.....	48	39	47	.....
24	82	84	.....	.....	44	.....	46	.....	43	45	.....	60	43	58	55	57	58	68	.....	76	.....	48	39	48	.....
25	82	.....	84	.....	.....	.....	46	.....	44	.....	60	60	42	58	54	55	58	69	.....	76	.....	46	40	48	.....
26	83	.....	84	.....	.....	44	46	.....	44	.....	62	61	43	56	54	.....	59	70	.....	76	.....	46	40	48	.....
27	84	.....	83	.....	.....	44	46	.....	.....	47	62	.....	42	56	55	.....	60	70	.....	76	.....	46	39	50	.....
28	.....	84	83	.....	.....	45	.....	.....	.....	48	64	.....	42	53	56	.....	60	70	.....	76	.....	45	39	50	.....
29	.....	84	83	.....	45	45	.....	.....	44	48	64	.....	42	.....	56	.....	61	70	.....	76	.....	45	40	.....	.....
30	84	85	.....	.....	45	45	.....	.....	.....	48	.....	.....	43	.....	56	.....	61	70	.....	76	.....	45	40	.....	.....
31	.....	85	.....	.....	.....	.....	.....	.....	.....	48	.....	.....	43	.....	.....	.....	.....	70	.....	.....	.....	46	40	.....	.....

[1] Temperatures observed in river from survey boat. Other temperatures recorded by Eagle Street Plant, Sewerage and Water Board, New Orleans, La.

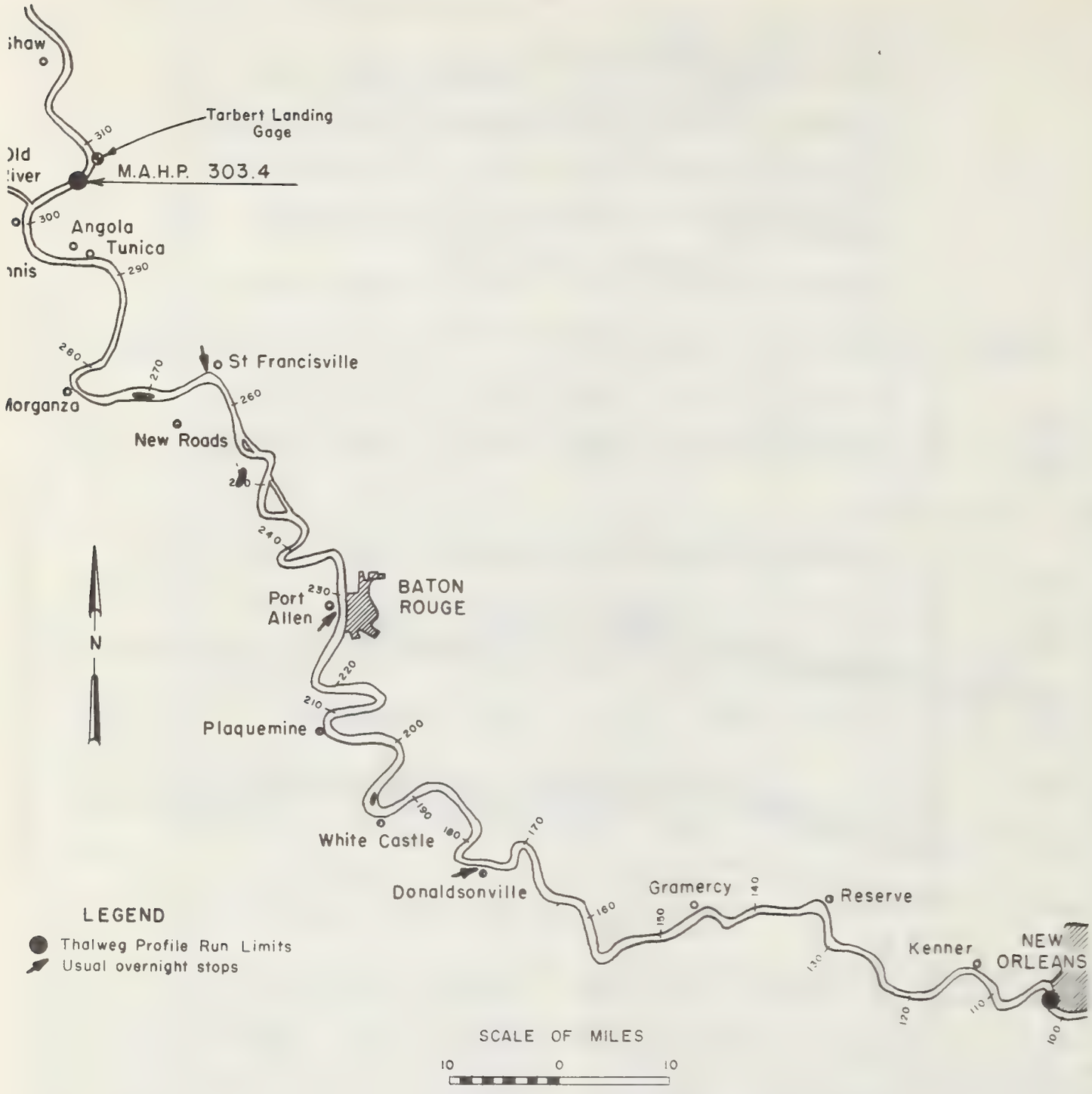

FIGURE 2. — Reach of river on which seven thalweg profile runs were made, 1958-62.

vations must be made when and as feasible. Some of the riverbed data were obtained by using thalweg profile microfilms in a "viewer"; a much more convenient technique than using the fathometer rolls or reproductions of same.

The resulting riverbed data are tabulated in two ways:

(1) Tables 2 to 7: a detailed tabulation of comparative measurements and physical characteristics of the crossings at the times of the several thalweg surveys. Study of the stage and discharge hydrographs (figs. 3, 4, 5, 6, and 7) is suggested as a preliminary review to a study of the tables.

The thalweg run in February 1959 was made at a stage substantially (6 or more feet) lower than other runs, and the data are accordingly not strictly comparable. The lowerings of the crossings and the reductions in sand wave amplitudes in tables 2 and 8 would have been substantially greater if the February 1959 run had been made at the same nominal stage (37

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)* [1]

Crossing location mile, A.H.P.,[2] date, and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Feb. 1959, upstream.	−68	.....	(4)	.....	No definite waves. Smooth undulations.	Upper end smooth with undulations. 5 ft. waves at lower end.	Top smooth with general undulations.
110.0, Feb. 1959, downstream.	−66	−5.5	Amplitude 8, length 190.	−5.5	Smooth with some general undulations.	Upper end waves 4 to 6 ft. Smooth to lower end.	Top smooth with general undulations.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough. Waves 4 to 6 ft.	Rough waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Feb. 1959, upstream.	−44.0	.....	Amplitude 2, length 35.	.....	Lower end smooth. Upper end waves 1 to 2 ft.	Smooth with some very gentle undulations.	Top smooth. Some gentle undulations.
116.0, Feb. 1959, downstream.	−45.0	−4.25	Amplitude 4, length 70.	−4.5	Lower end waves 2 ft. Upper end waves 4 ft.	Smooth	Top smooth and regular.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Feb. 1959, upstream.	−59.0	.....	(5)	.....	Smooth.	Smooth.	Top smooth.
140.5, Feb. 1959, downstream.	−58.0	−2.75	(5)	−5.5	Smooth. Slightly rough upstream end.	Smooth.	Top smooth.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Feb. 1959, upstream.	−37.5	.....	Amplitude 3, length 75.	.....	Smooth nearly to top. Then waves 1 to 2 ft.	Smooth	Upstream half with waves 1 to 3 ft. Downstream half smooth.
154.0, Feb. 1959, downstream.	−34.0	−1.50	Amplitude 4, length 100.	−3.0	Rough. Waves 3 to 4 ft.	Smooth.	Rough except for upstream quarter which is smooth. Waves 2 to 3 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Feb. 1959, upstream.	−45.5	.....	Amplitude 4, length 125.	.....	Smooth.	Rough except at middle. Waves 3 ft.	Top smooth upstream half. Rough downstream half. Waves 2 to 4 ft.

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)*[1]*—Continued*

Crossing location mile, A.H.P.,[2] date and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
163.0, Feb. 1959, downstream.	−51.5	−2.0	Amplitude 2, length 60.	−5.5	Smooth except downstream third. Waves 2 ft.	Smooth except 4 ft. waves upstream toe.	Top rough. Waves 2 to 3 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Feb. 1959. upstream.	−55.0	.....	Amplitude 8, length 150.	.....	Rough 5 to 7 ft. Smoothing up at bottom.	Smooth.	Top rough. Waves 5 to 8 ft. Downstream end smooth.
167.0, Feb. 1959, downstream.	−53.5	+ .5	Amplitude 10, length 250.	−2.5	Rough. Waves 3 to 6 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smoo h.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Feb. 1959, upstream.	−46.0	.....	Amplitude 14, length 180.	.....	Rough. Waves 6 to 9 ft.	Smooth. Very steep.	Smooth.
183.0, Feb. 1959, downstream.	−41.5	−1.25	Amplitude 11, length 250.	−1.5	Rough. Waves 3 to 10 ft.	Rough. Waves 6 to 11 ft.	Smooth except up and downstream ends with waves 3 to 6 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Feb. 1959, upstream.	−38.0	.....	Amplitude 7, length 160.	.....	Smooth at bottom. Waves 3 to 7 ft. at top.	Smooth.	Top smooth except short stretch upstream with waves 4 to 5 ft.
190.5, Feb. 1959, downstream.	−37.5	−2.50	Amplitude 7, length 120.	−4.5	Smooth at bottom. Waves 3 to 5 ft. at top.	Smooth.	Top smooth except short stretch upstream with waves 3 to 7 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Feb. 1959, upstream.	−39.5	.....	Amplitude 8, length 175.	.....	Rough. Waves 3 to 8 ft.	Smooth.	Top smooth except small part upstream end waves 5 to 8 ft.
197.5, Feb. 1959, downstream.	−37.5	−1.75	Amplitude 7, length 175.	−2.0	Rough. Waves 1 to 2 ft.	Smooth.	Top mostly smooth. Upstream end waves 2 to 7 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Feb. 1959, upstream.	−40.0	.....	Amplitude 8, length 130.	.....	Rough. Waves 3 to 9 ft.	Smooth except few 2 to 3 ft. waves at bottom.	Top smooth, except few waves 2 to 5 ft. at upstream end.

204.0, Feb. 1959, downstream.	−37.5	−5.0	Amplitude 7, length 140.	−3.5	Bottom smooth. Waves 2 to 7 ft. to top.	Smooth.	Top smooth, except upstream end, waves 2 to 7 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Feb. 1959, upstream.	−41.5	.....	Amplitude 9, length 105.	.....	Rough. Waves 5 to 9 ft.	Smooth.	Top smooth, except waves 2 to 7 ft. at upstream end.
212.0, Feb. 1959, downstream.	−41.5	−4.2	Amplitude 9, length 140.	−5.5	Rough. Waves 4 to 9 ft.	Smooth.	Top smooth, except upstream third with waves 2 to 7 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Feb. 1959, upstream.	−33.5	.....	Amplitude 13, length 500.	.....	Smooth with undulations at bottom.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 13 ft.
223.5, Feb. 1959, downstream.	−33.5	−2.0	Amplitude 13, length 500.	−1.0	Rough. Waves 5 to 11 ft.	Rough. Waves 4 to 11 ft.	Rough. Waves 5 to 13 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Feb. 1959, upstream.	−33.0	.....	Amplitude 17, length 330.	.....	Smooth.	Smooth with undulations.	Rough. Waves 7 to 17 ft. Beginning to smooth out.
236.0, Feb. 1959, downstream.	−28.5	−5.50	Amplitude 9, length 100.	−2.0	Rough. Waves 3 to 6 ft.	Lower part smooth. Waves 5 to 9 ft.	Rough. Waves 2 to 6 ft. Beginning to smooth out.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Feb. 1959, upstream.	−25.0	.....	Amplitude 12, length 300.	.....	Smooth.	Smooth.	Smooth except downstream quarter with waves 5 to 12 ft.
241.0, Feb. 1959, downstream.	−24.0	−3.0	Amplitude 12, length 350.	− .5	Regular with waves 2 to 3 ft.	Lower part smooth. Waves 5 to 10 ft.	Rough. Waves 4 to 12 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Feb. 1959, upstream.	−19.5	.....	Amplitude 11, length 175.	.....	Lower part smooth. Upper part waves 3 to 6 ft.	Rough. Waves 4 to 11 ft.	Downstream quarter is smooth. Remainder waves 4 to 8 ft.
251.0, Feb. 1959, downstream.	−20.0	−5.25	Amplitude 8, length 75.	−4.5	Lower part smooth. Upper part waves 3 to 8 ft.	Rough. Waves 3 to ? ft.	Rough. Waves 3 to 7 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.

See footnotes at end of table.

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)*[1]*—Continued*

Crossing location mile, A.H.P.,[2] date, and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
258.5, Feb. 1959, upstream.	−21.0	.....	Amplitude 12, length 350.	.....	Regular with waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 3 to 12 ft.
258.5, Feb. 1959, downstream.	−19.5	−1.25	Amplitude 11, length 80.	−4.0	Regular with small waves 2 to 3 ft.	Rough. Waves.	Rough. Waves 6 to 11 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Feb. 1959, upstream.	−33.5	.....	Amplitude 11, length 225.	.....	Rough.	Smooth at bottom. Top with waves 5 to 11 ft.	Upstream end smooth. Downstream with waves 3 to 8 ft.
269.5, Feb. 1959, downstream.	−37.5	−0.75	Amplitude 11, length 275.	−2.5	Rough. Waves 3 to 9 ft.	Smooth with undulations.	Rough. Waves 3 to 11 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Feb. 1959, upstream.	−16.5	.....	Amplitude 13, length 350.	.....	Rough Waves 3 to 5 ft.	Rough. Waves 2 to 3 ft.	Rough. Waves 3 to 13 ft.
273.0, Feb. 1959, downstream.	−18.5	−9.5	Amplitude 8, length 195.	−8.0	Rough Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Feb. 1959, upstream.	−27.5	.....	Amplitude 10.[6]	.....	Smooth.	Rough. Undulations 3 to 6 ft.	Smooth.
279.0, Feb. 1959, downstream.	−32.0	+1.0	Amplitude 6, length 175.	−14.5	Rough at bottom third. Remainder smooth.	Smooth and steep.	Smooth.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Feb. 1959, upstream.	−23.5	.....	Amplitude 10, length 160.	.....	Smooth.	Rough. Waves 3 to 10 ft.	Rough downstream two-thirds waves 3 to 8 ft. Upstream third smooth.
286.5, Feb. 1959, downstream.	−24.0	−17.25	Amplitude 8, length 125.	−16.0	Smooth.	Rough. Waves 3 to 8 ft.	Rough. Small waves 1 to 3 ft.
298.0, Aug. 1958, upstream.	− 0.5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.

298.0, Feb. 1959, upstream.	— 7.5	.....	Amplitude 12, length 375.	.....	Rough. Waves 3 to 6 ft.	Waves 2 to 3 ft. Steep.	Rough. Waves 3 to 10 ft.
298.0, Feb. 1959, downstream.	— 7.5	—6.5	Amplitude 6, length 250.	—1.5	Rough. Waves 3 to 6 ft.	Waves 1 to 2 ft. Steep.	Rough. Waves 2 to 6 ft. Upstream third beginning to smooth out.

[1]Because of the lower stage for the Feb. 9–18, 1959, run (see hydrographs), the differences in heights of crossings and wave amplitudes are only partially ascribable to temperature.
[2]Above head of passes.
[3]Mean low water.
[4]No definite waves. Smooth undulations.
[5]Smooth.
[6]No waves.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	—63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upstream end rough. Waves 3 to 5 ft. Downstream part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	—60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Apr. 1960, upstream.	—65.0	.....	Amplitude 6, length 190.	.....	Rough. Waves 2 to 5 ft.	Rough. Waves 4 to 9 ft.	Top rough. Waves 3 to 6 ft.
110.0, Apr. 1960, downstream.	—64.0	—3.0	Amplitude 9, length 250.	—2.00	Smooth. Waves 2 to 4 ft.	Rough. Waves 3 to 10 ft. Steep drop at downstream end.	Top rough. Waves 4 to 9 ft.
116.0, Aug. 1958, upstream.	—41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upstream end. Then waves 2 to 3 ft.	Upstream end, waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	—39.5	.....	Amplitude 8, length 165.	.....	Downstream end undulating. Upstream end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Apr. 1960, upstream.	—42.5	.....	Amplitude 8, length 150.	.....	Smooth with slight ripples.	Smooth and steep.	Top rough. Waves 2 to 8 ft.
116.0, Apr. 1960, downstream.	—40.0	—1.00	Amplitude 6, length 100.	— .50	Rough. Waves 2 to 6 ft.	Rough. Waves 4 to 10 ft.	Top rough. Waves 2 to 6 ft.
140.5, Aug. 1958, upstream.	—54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upstream end rough. Waves 2 to 4 ft. Downstream end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	—57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end, waves 2 to 5 ft.	Upstream end rough. Waves 2 to 3 ft. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Apr. 1960, upstream.	—52.5	.....	Amplitude 9, length 140.	.....	Smooth at bottom. Rough at top. Waves 2 to 5 ft.	Rough. Waves 2 to 13 ft.	Top rough. Waves 3 to 9 ft.
140.5, Apr. 1960, downstream.	—51.0	+4.0	Amplitude 11, length 250.	+4.50	Rough. Waves 1 to 6 ft.	Rough. Waves 4 to 8 ft.	Rop rough. Waves 2 to 11 ft.

See footnotes at end of table.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Apr. 1960, upstream.	−38.0	.....	Amplitude 8, length 180.	.....	Smooth. Small waves 1 to 2 ft.	Smooth, steep downstream drop.	Extreme upstream end rough, waves 1 to 8 ft. Middle portion and downstream end smooth with waves 1 to 2 ft.
154.0, Apr. 1960, downstream.	−34.5	−2.00	Amplitude 9, length 200.	+2.00	Rough. Waves 3 to 5 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 4 to 9 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth downstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Apr. 1960, upstream.	−47.0	.....	Amplitude 6, length 130.	.....	Rough. Waves 2 to 5 ft.	Rough. Waves 2 to 10 ft. Steep slope.	Top rough. Waves 3 to 6 ft.
163.0, Apr. 1960, downstream.	−46.0	−0.25	Amplitude 9, length 180.	−1.00	Rough. Waves 4 to 17 ft.	Smooth, steep slope with slight ripples 1 ft.	Top rough. Waves 4 to 9 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants of large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Apr. 1960, upstream.	−60.5	.....	Amplitude 11, length 270.	.....	Rough. Small waves 2 to 4 ft.	Rough. Waves 4 to 12 ft.	Top rough. Waves 2 to 11 ft.
167.0, Apr. 1960, downstream.	−57.0	−4.00	Amplitude 9, length 300.	−1.50	Smooth with ripples 1 to 3 ft.	Rough. Waves 4 to 12 ft.	Top rough. Waves 4 to 9 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Apr. 1960, upstream.	−47.5	.....	Amplitude 19, length 550.	.....	Rough. Waves 4 to 17 ft.	Rough. Waves 4 to 22 ft.	Rough. Waves 4 to 19 ft.
183.0, Apr. 1960, downstream.	−45.0	−3.75	Amplitude 11, length 550.	+1.00	Rough. Waves 4 to 15 ft.	Rough. Waves 3 to 17 ft.	Top rough. Waves 3 to 11 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Apr. 1960, upstream.	−45.0	.....	Amplitude 10, length 400.	.....	Rough. Waves 3 to 8 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 4 to 10 ft.
190.5, Apr. 1960, downstream.	−40.0	−7.50	Amplitude 14, length 500.	− .50	Rough. Waves 2 to 7 ft.	Rough. Waves 3 to 14 ft.	Top rough. Waves 3 to 14 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.

197.5, Apr. 1960, upstream.	−38.0	.....	Amplitude 6, length 300.	.....	Smooth at bottom. Rough with waves 2 to 5 ft. at top.	Rough. Waves 2 to 4 ft.	Top rough. Waves 1 to 6 ft.
197.5, Apr. 1960, downstream.	−35.5	−0.00	Amplitude 7, length 400.	−3.00	Smooth at bottom with 1 ft. ripples. Rough at top.	Rough. Waves 3 to 11 ft.	Top rough with waves 3 to 7 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Apr. 1960, upstream.	−39.5	.....	Amplitude 12, length 300.	.....	Rough. Waves 3 to 15 ft.	Smooth and steep. Long undulations.	Top rough. Waves 4 to 12 ft.
204.0, Apr. 1960, downstream.	−38.0	−5.00	Amplitude 14, length 350.	+2.00	Rough. Waves 2 to 6 ft.	Smooth, steep slope with 1 ft. ripples.	Top rough. Waves 3 to 14 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 6 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 7 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Apr. 1960, upstream.	−48.0	.....	Amplitude 8, length 200.	.....	Rough. Waves 2 to 4 ft.	Rough. Waves 3 to 10 ft.	Top rough. Waves 2 to 8 ft.
212.0, Apr. 1960, downstream.	−48.0	−10.75	Amplitude 11, length 250	−5.0	Smooth.	Smooth at bottom. Rough at top.	Top rough. Waves 2 to 11 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Apr. 1960, upstream.	−26.0	.....	Amplitude 10, length 370.	.....	Smooth 1 ft. ripples.	Rough. Waves 3 to 10 ft.	Rough. Waves 3 to 10 ft.
223.5, Apr. 1960, downstream.	−32.0	+2.50	Amplitude 7, length 150.	−5.50	Rough. Waves 3 to 6 ft.	Rough. Waves 2 to 10 ft.	Rough. Waves 3 to 7 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth. Waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Apr. 1960, upstream.	−34.0	.....	Amplitude 21, length 1,070.	.....	Upstream waves 3 to 14 ft. Downstream waves 1 to 5 ft.	Rough waves upstream 3 to 18 ft. Downstream smooth, steep drop.	Very long undulations 5 to 21 ft.
236.0, Apr. 1960, downstream.	−34.0	−8.75	Amplitude 24, length 1,280.	+8.00	Rough waves 3 to 19 ft.	Rough waves, steep drop.	Very long undulations, 5 to 24 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upstream part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upstream part, with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Apr. 1960, upstream.	−27.0	.....	Amplitude[3] length	.....	Rough. Waves 3 to 6 ft.	Smooth upstream. Sudden drop, then rough waves 3 to 8 ft.	Smooth with slight ripples 1 ft.
241.0, Apr. 1960, downstream.	−25.0	−4.50	Amplitude[3] length	−12.00	Rough. Waves 2 to 7 ft.	Smooth, sudden drop.	Smooth with slight ripples 1 ft.

See footnotes at end of table.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Apr. 1960, upstream.	−20.5	.....	Amplitude 12, length 300.	.....	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 7 ft.	Rough. Waves 4 to 12 ft.
251.0, Apr. 1960, downstream.	−20.0	−5.75	Amplitude 12, length 250.	−2.00	Rough. Waves 3 to 8 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 4 to 12ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, Apr. 1960, upstream.	−29.0	.....	Amplitude[3] length	.....	Rough. Waves 2 to 7 ft.	Smooth with small waves.	Smooth.
258.5, Apr. 1960, downstream.	−27.0	−9.00	Amplitude[3] length	−15.50	Rough with waves 2 to 6 ft.	Rough. Waves 2 to 5 ft.	Very smooth.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough. Waves 3 to 10 ft. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough, with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Apr. 1960, upstream	−40.0	.....	Amplitude 5, length 75.	.....	Rough. Waves 2 to 4 ft. Steep slope.	Rough. Waves 3 to 6 ft.	Small waves 1 to 5 ft.
269.5, Apr. 1960, downstream.	−43.5	−7.00	Amplitude 9, length 250.	−6.50	Rough. Waves 2 to 6 ft.	Top rough with 1 to 6 ft. waves. Bottom smooth.	Rough. Waves 3 to 9 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth, with long undulations.	Rough upstream end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Apr. 1960, upstream.	−15.0	.....	Amplitude[3] length	.....	Smooth upward slope.	Smooth steep slope.	Very smooth at top.
273.0, Apr. 1960, downstream.	−12.5	−5.75	Amplitude[3] length[4]	−18.50	Smooth with ripples 2 ft.	Smooth with long undulations.	Very smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Apr. 1960, upstream.	−26.5	.....	Amplitude 8, length 400.	.....	Rough. Waves 3 to 5 ft.	Smooth steep slope.	Rough. Waves 4 to 8 ft.
279.0, Apr. 1960, downstream.	−22.0	+6.50	Amplitude 8, length 540.	−14.50	Rough. Waves 3 to 6 ft.	Smooth steep slope.	Rough. Waves 4 to 8 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.

286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Apr. 1960, upstream.	−10.0	.....	Amplitude 15, length 2,000	.....	Rough. Waves 2 to 13 ft.	Smooth.	Very long, smooth waves, 7 to 15 ft.
286.5, Apr. 1960, downstream.	− 8.0	−2.50	Amplitude 25, length 2,400.	−5.00	Rough. Waves 3 to 10 ft.	Smooth with slight ripples.	Very smooth surface between 4 long waves, 13 to 25 ft. in amplitude and 1,200 to 2,400 ft. in length.
298.0, Aug. 1958, upstream.	+ 0.5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Apr. 1960, upstream.	− 8.5	.....	Amplitude 10, length 350.	.....	Smooth, ripples 1 ft.	Smooth and steep.	Smooth with waves 1 to 2 ft.
298.0, Apr. 1960, downstream.	− 9.0	−7.75	Amplitude 9, length 550.	−1.00	Smooth, ripples 1 ft.	Ripples and steep.	One to four foot waves at upstream end. Rough, with waves 1 to 9 ft. on downstream end.
286.5, upstream.	.....	.....	Amplitude 26, length 1,600.	THESE WERE THE LARGEST WAVES FOUND ANYWHERE.			
286.5, downstream.	.....	.....	Amplitude 26, length 2,200.	SIMILAR WAVES FOUND AT THIS LOCATION IN 1958.			

[1] Above head of passes.
[2] Mean low water.
[3] Smooth.
[4] No waves.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)*

Crossing location mile A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 4 to 13 ft.	Almost smooth throughout	Top rough. Waves 6 to 13 ft.
110.0, Mar. 1961, upstream.	−70.0	.....	([3])	.....	Generally smooth.	Generally smooth.	Large irregularities— amplitude less than 5 ft.
110.0, Mar. 1961, downstream.	−71.0	−9.00	([3])	.....	Ripply.	Generally rough. Steep drop.	Large irregularities—amplitude less than 5 ft.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough waves 4 to 6 ft.	Rough waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Mar. 1961, upstream.	−49.0	.....	Amplitude 8, length 100.	.....	Waves 2 to 5 ft. Rough.	Rough. Waves 3 to 8 ft.	Rough. Waves 3 to 7 ft.

See footnotes at end of table.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)—Continued*

Crossing location mile A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
116.0, Mar. 1961, downstream.	−49.0	−8.75	Amplitude 8, length 200.	+0.50	Rough. Waves 3 to 6 ft.	Rough. Small waves 1 to 3 ft.	Rough. Waves 2 to 5 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Mar. 1961, upstream.	−52.0	.....	Amplitude 9, length 150.	.....	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 8 ft.	Rough. Waves 3 to 8 ft.
140.5, Mar. 1961, downstream.	−54.0	+2.75	Amplitude 11, length 500.	+4.50	Rough. Waves 2 to 10 ft.	Rough. Waves 5 to 10 ft.	Rough. Waves 5 to 10 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Mar. 1961, upstream.	−39.0	.....	Amplitude 8, length 100.	.....	Rough. Waves 9 to 100 ft.	Rough. Waves 10 to 150 ft.	Downstream part has a few waves. Top is smooth. Upstream end has a few waves.
154.0, Mar. 1961, downstream.	−36.0	−3.25	Amplitude 22, length 225.	+8.50	Rough. Waves 10 to 20 ft. upstream end.	Rough. Waves 3 to 10 ft.	Rough. Small waves 2 to 4 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Mar. 1961, upstream.	−47.0	.....	Amplitude 12, length 100.	.....	Rough. Waves 5 to 12 ft.	Gentle rounded waves.	Rough waves 7 x 125 ft. Waves 4 to 8 ft.
163.0, Mar. 1961, downstream.	−53.0	−3.75	Amplitude 15, length 300.	+5.00	Rough. Waves 10 to 15 ft.	Steep and rough. Waves 1 to 5 ft.	Rough. Waves 10 to 15 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Mar. 1961, upstream.	−58.0	.....	Amplitude 7, length 170.	.....	Rough. Waves 3 to 7 ft. Waves 100 ft. long.	Rough. Waves long and flat.	Generally smooth. Very slight undulations—less than 1 ft. amplitude.
167.0, Mar. 1961, downstream.	−47.0	+2.25	Amplitude 15, length 500.	− .50	Generally smooth.	Smooth. Gets rougher at lower end. Waves 5 to 9 ft.	Rough. Large waves.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.

183.0, Mar. 1961, upstream.	−46.0	.....	Amplitude 12, length 125.	.....	Generally smooth. Long and flat. Very few waves. Waves 4 to 7 ft.	Rough. Sloping and erratic. Waves 15 x 200 ft.	Generally smooth. A little rough on each edge.
283.0, Mar. 1961, downstream.	−41.0	−1.00	Amplitude 14, length 220.	−1.00	Rough. Upstream end gets smooth. Waves 1 to 6 ft.	Rough. Waves 5 to 14 ft.	Smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Mar. 1961, upstream.	−44.0	.....	Amplitude 15, length 400.	.....	Rough. Small waves 2 to 5 ft.	Rough. Waves 6 to 15 ft.	Rough. Large waves 8 x 300 ft.
190.5, Mar. 1961, downstream.	−41.0	−7.25	Amplitude 18, length 400.	+5.00	Rough. Waves 5 to 10 ft.	Rough. Waves 2 to 8 ft.	Rough. Large waves.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Mar. 1961, upstream.	−43.0	.....	Amplitude 8, length 300.	.....	Rough. Well defined undulations—2 to 8 ft. Mostly about 4 ft.	Rough. Waves 1 to 7 ft.	Short. Generally smooth top. About 1,000 ft. long.
197.5, Mar. 1961, downstream.	−40.0	−4.75	Amplitude 10, length 250.	−0.50	Rough. 2 to 10 ft. waves.	Rough. Waves 5 to 10 ft.	Rough. Large waves.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Mar. 1961, upstream.	−44.0	.....	Amplitude 4, length 200.	.....	Rough. No well developed wave system.	Rough with terrace effect.	Fairly smooth—no waves.
204.0, Mar. 1961, downstream.	−50.0	−13.25	Amplitude 11, length 300.	−3.50	Smooth. Gentle undulations.	Rough. Steep drop.	Smooth—gentle undulations.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Mar. 1961, upstream.	−38.0	.....	Amplitude 10, length 200.	.....	Rough. Waves 5 to 10 ft.	Rough. Not well defined.	Short top with steep approaches in the form of large waves 1,600 ft. long.
212.0, Mar. 1961, downstream.	−46.0	−4.75	Amplitude 8, length 200.	−5.50	Rough. Waves 2 to 8 ft.	Rough. Waves 2 to 5 ft.	Smooth. Gentle undulations.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Mar. 1961. upstream.	−53.0	.....	Amplitude 15, length 600.	.....	Rough. Waves 2 to 10 ft.	Rough. Big waves over 500 wide x 15 ft. high.	Flat. Smooth top.
223.5, Mar. 1961, downstream.	−51.0	−20.50	Amplitude 14, length 350.	+ .50	Rough. Waves 2 to 5 ft.	Rough. Big waves.	Flat. Smooth.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.

See footnotes at end of table.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Mar. 1961, upstream.	−46.0	.....	Amplitude 5, length 150.	.....	Rough. Varying to smooth and ripply. Waves 5 x 150 ft.	Smooth and ripply. No wave form. Steep drop.	Smooth with slight ripples 1 ft. or less.
236.0, Mar. 1961, downstream.	−47.0	−21.25	Amplitude 4, length 150.	−10.5	Rough. 2 to 4 ft. waves.	Smooth. Steel drop.	Smooth. Gentle undulations.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Mar. 1961, upstream.	−35.0	.....	Amplitude 10, length 125.	.....	Very rough. Well developed waves 5 to 10 ft.	Slightly rough. Steep drop .	Short and less rough than approach. No wave form.
241.0, Mar. 1961, downstream.	−37.0	−14.50	Amplitude 10, length 150.	−1.50	Rough. Waves 10 ft.	Smooth. Steep drop. Gentle undulations.	Smooth on top.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Mar. 1961, upstream.	−30.5	.....	Amplitude 15, length 125.	.....	Very rough. Waves 2 to 8 ft.	Rough. Waves 15 x 125 ft.	Short and smooth. Rough on both approaches. Downstream waves 3 to 9 ft.
251.0, Mar. 1961, downstream.	−32.0	−16.75	Amplitude 13, length 300.	± .00	Rough. Small waves upstream part. Larger waves downstream part.	Rough. Large waves 12 x 300 ft.	Rough. Large waves.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, Mar. 1961, upstream.	−28.0	.....	Amplitude 18, length 400	.....	Very long, varying from flat to very rough. Waves 5 to 15 ft.	Fairly smooth. Undulations. Rough with steep drop at end.	Very large waves 17 x 700 ft.
258.5, Mar. 1961, downstream.	−25.5	−7.75	Amplitude 16, length 300.	+1.50	Very long. Rough. Waves very large.	Fairly smooth undulations. Steep drop at end.	Rough. Large waves 15 x 300 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Mar. 1961, upstream.	−49.5	.....	Amplitude 15, length 300.	.....	Rough. Small waves 2 to 5 ft.	Rough. Waves 5 to 15 ft.	Rough. Waves 5 to 7 ft.
269.5, Mar. 1961, downstream.	−46.0	−13.00	Amplitude 9, length 275.	−1.50	Rough. Small waves 1 to 3 ft.	Smooth. Generally undulating.	Rough. Waves 3 to 9 ft.

273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Mar. 1961, upstream.	−26.5	.....	Amplitude 7, length 200.	.....	Rough. Waves 3 to 8 ft.	Smooth.	Smooth.
273.0, Mar. 1961, downstream.	−29.0	−19.75	Amplitude 15, length 300.	−7.50	Rough. Waves 3 to 8 ft. Upper part generally smooth.	Rough. Waves 5 to 15 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Mar. 1961, upstream.	−39.0	.....	Amplitude 15, length 400.	.....	Rough. Waves 9 ft.	Smooth undulations.	Rough. Large waves 15 x 400 ft.
279.0, Mar. 1961, downstream.	−35.0	−6.25	Amplitude 15, length 550.	−7.50	Rough. Waves 3 to 8 ft.	Smooth upper part. Lower part rough. Waves 2 to 4 ft.	Rough. Smooth upper part. Lower part rough.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Mar. 1961, upstream.	−31.0	.....	Amplitude 6, length 100.	.....	Rough. Small waves 2 to 4 ft.	Waves at lower end, 3 to 6 ft.	Top generally smooth.
286.5, Mar. 1961, downstream.	−30.0	−24.0	Amplitude 8, length 200.	−18.00	Rough. Large waves 4 to 8 ft.	Smooth at upstream end. Very rough downstream end. Waves 3 to 6 ft.	Upstream part rough. Downstream part smooth.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Mar. 1961, upstream.	−18.0	.....	Amplitude 8, length 125.	.....	Rough. Small waves 1 to 3 ft.	Rough. Steep. Larger waves at bottom. Waves 1 to 4 ft.	Rough. Waves 3 to 8 ft.
298.0, Mar. 1961, downstream.	−25.0	−20.50	Amplitude 11, length 150.	−1.00	Rough. Waves 3 to 11 ft.	Rough. Small waves at upper end. General undulations.	Rough. Waves 2 to 6 ft.

[1] Above head of passes.
[2] Mean low water.
[3] No wave system.

TABLE 5.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus May 8 to 12, 1961 (64° to 66° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, May 1961, upstream.	−70.0	.....	Amplitude 15, length 300.	.....	Rough. Waves 2 to 4 ft.	Rough. Waves 10 to 15 ft.	Rough. Waves 2 to 15 ft.
110.0, May 1961, downstream.	−72.0	−9.50	Amplitude 7, length 100.	+1.50	Rough. Waves 1 to 3 ft.	Rough. Waves 4 to 7 ft.	Smooth—ripply.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, May 1961, upstream.	−36.0	.....	Amplitude 12, length 300.	.....	Rough. Waves 1 to 8 ft.	Upper end smooth. Lower end rough.	Rough. Waves 3 to 6 ft.
116.0, May 1961, downstream.	−36 0	+4.25	Amplitude 15, length 300.	+6.00	Ripples to rough. Waves 3 to 10 ft.	Ripples.	Rough. Waves 3 to 15 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, May 1961, upstream.	−48.0	.....	Amplitude 17, length 500.	.....	Rough. Waves 2 to 8 ft.	Rough. Waves 1 to 6 ft.	Rough. Waves 5 to 15 ft.
140.5, May 1961, downstream.	−49.0	+7.25	Amplitude 10, length 300.	+8.00	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, May 1961, upstream.	−27.0	.....	Amplitude 12, length 250.	.....	Rough. Ripply.	Rough at lower end. Waves 2 to 4 ft. Short and steep.	Rough. Waves 2 to 12 ft.
154.0, May 1961, downstream.	−25.0	+8.25	Amplitude 13, length 1,300.	+6.00	Ripply.	Rough. Waves 2 to 5 ft.	Rough. Waves 2 to 10 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Rop rough. Waves 3 to 6 ft.
163.0, May 1961, upstream.	−42.0	.....	Amplitude 8, length 250.	.....	Rough. Waves 3 to 5 ft.	Rough. Irregular. Short and steep.	Rough. Waves 3 to 15 ft.
163.0, May 1961, downstream.	−43.0	+3.75	Amplitude 8, length 300.	−0.50	Rough. Waves 3 to 6 ft.	Smooth. Steep.	Rough. Waves 3 to 10 ft.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, May 1961, upstream.	−46.0	.....	Amplitude 12, length 500.	.....	Rough. Small waves 2 to 3 ft.	Rough. Very large waves 12 ft.	Rough. Waves 2 to 8 ft.
167.0, May 1961, downstream.	−48.0	+7.75	Amplitude 20, length 500.	+4.50	Rough. Large waves 5 to 15 ft.	Ripply and steep.	Rough. Waves 7 to 20 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, May 1961, upstream.	−44.0	.....	Amplitude 15, length 650.	.....	Long—rough.	Smooth. Gentle undulations.	Gentle undulations.
183.0, May 1961, downstream.	−47.5	−3.25	Amplitude 25, length 750.	+6.00	Rough. Large waves.	Rough. Waves 2 to 4 ft.	Smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, May 1961, upstream.	−32.0	.....	Amplitude 9, length 300.	.....	Long, gently undulating.	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 8 ft.
190.5, May 1961, downstream.	−32.0	+3.25	Amplitude 18, length 500.	+2.00	Rough. Waves 2 to 10 ft.	Long. Smooth and undulating.	Rough. Large waves.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, May 1961, upstream.	−30.0	.....	Amplitude 20, length 1,100.	.....	Rough. Waves 2 to 10 ft.	Irregular, steep, no order to surface.	Top rough. Waves 1 to 13 ft.
197.5, May 1961, downstream.	−29.0	+7.25	Amplitude 20, length 750.	+10.5	Rough. Waves 2 to 15 ft.	Rough. Waves 2 to 4 ft.	Rough. Large waves 5 to 15 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, May 1961, upstream.	−31.0	.....	Amplitude 14, length 800.	.....	Rough. Ripples at start. Waves 7 to 12 ft. near top.	Steep. Undulations.	Rough. Waves 5 to 12 ft.
204.0, May 1961, downstream.	−33.0	+1.75	Amplitude 14, length 500.	+3.00	Rough. Waves 5 to 15 ft.	Ripply.	Rough. Waves 5 to 15 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, May 1961, upstream.	−29.0	.....	Amplitude 23, length 1,100.	.....	Rough. Waves 4 to 15 ft.	Rough. Waves 1 to 11 ft.	Few large smooth waves 6 to 12 ft.
212.0, May 1961, downstream.	−27.0	+9.25	Amplitude 24, length 1,000	+9.00	Rough. Waves 2 to 15 ft.	Rough. Waves 1 to 10 ft.	Rough. Large waves 10 to 20 ft.

See footnotes at end of table.

TABLE 5.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus May 8 to 12, 1961 (64° to 66° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, May 1961, upstream.	−26.0	.....	Amplitude 12, length 900.	.....	Rough. Large waves 5 to 15 ft.	Undulating. Smooth.	Rough. Large waves.
223.5, May 1961, downstream.	−36.5	+ .25	Amplitude 18, length 300.	+1.00	Rough. Large waves 2 to 15 ft.	Ripply.	Smooth. Large waves.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, May 1961, upstream.	−28.0	.....	Amplitude 14, length 350.	.....	Rough. Waves 1 to 10 ft.	Undulating. Steep drop.	Rough. Waves 7 to 14 ft.
236.0, May 1961, downstream.	−32.0	−4.75	Amplitude 10, length 400.	−3.00	Rough. Waves 2 to 10 ft.	Rough. Steep. Large waves.	Rough. Large waves 4 to 12 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, May 1961, upstream.	−36.0	.....	Amplitude 8, length 100.	.....	Rough. Ripply.	Rough. Steep descent.	Rough. Waves 2 to 8 ft.
241.0, May 1961, downstream.	−34.0	−13.50	Amplitude 10, length 250.	−2.50	Ripples to rough. Waves 1 to 10 ft.	Rough. Steep descent.	Rough. Waves 3 to 10 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, May 1961, upstream.	−12.0	.....	Amplitude 10, length 150.	.....	Approach rough. Large waves 2 to 10 ft.	Smooth. Gentle undulations.	Rough. Frequent undulations.
251.0, May 1961, downstream.	−15.0	+1.00	Amplitude 13, length 500.	−2.50	Ripply to rough.	Undulating. Smooth.	Rough. Waves 2 to 4 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, May 1961, upstream.	−13.0	.....	Amplitude 13, length 200.	.....	Rough. Waves 2 to 8 ft.	Smooth. Undulating.	Rough. Waves 2 to 7 ft.
258.5, May 1961, downstream.	−32.0	−12.50	Amplitude 15, length 300.	−1.50	Rough. Waves 2 to 7 ft.	Smooth. Undulating.	Smooth. Undulating.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.

269.5, May 1961, upstream.	−51.0	.....	Amplitude 13, length 250.	.....	Steep. Rough. Irregular.	Rough. Waves 5 to 13 ft.	Rough. Waves 2 to 5 ft.
269.5, May 1961, downstream.	−53.0	−17.25	Amplitude 18, length 200.	+2.00	Steep. Rough. Irregular.	Rough. Waves 5 to 15 ft.	Rough. Waves 2 to 5 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, May 1961, upstream.	−18.0	.....	Amplitude 18, length 700.	.....	Ripply. Constant ascent.	Rough. Waves 2 to 5 ft.	Rough. Large waves.
273.0, May 1961, downstream.	−15.0	−8.50	Amplitude 10, length 150.	−4.50	Ripply.	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, May 1961, upstream.	−30.0	.....	Amplitude 20, length 400.	.....	Rough. Large waves.	Rough. Ripply. Steep. Waves 1 to 18 ft.	Rough. Waves 5 to 13 ft.
279.0, May 1961, downstream.	−34.5	−1.50	Amplitude 20, length 500.	−2.50	Rough. Waves 2 to 15 ft.	Smooth. Undulating.	Rough. Waves 5 to 20 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, May 1961, upstream.	−22.0	.....	Amplitude 13, length 1,000.	.....	Rough. Small waves 2 to 8 ft.	Rough. Irregular pattern.	Rough. Large waves.
286.5, May 1961, downstream.	−23.0	−16.0	Amplitude 17, length 800.	−10.00	Rough. Waves 17 x 400 ft.	Gentle. Undulating.	Rough. Large waves.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, May 1961, upstream.	−17.0	.....	[3]	.....	Ripply. Gentle undulations.	Ripply. Gentle undulations.	Ripply. Gentle undulations.
298.0, May 1961, downstream.	−16.0	−15.50	[3]	.....	Ripply. Gentle undulations.	Ripply. Gentle undulations.	Ripply. Gentle undulations.

[1] Above head of passes.
[2] Mean low water.
[3] No wave pattern.

TABLE 6.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus June 1 to 6, 1961 (70° to 73° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 2 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, June 1961, upstream.	−63.0	.....	Amplitude 22, length 394.	.....	Rough. Waves 2 to 10 ft. Steep approach slope.	Rough. Waves 1 to 6 ft.	Rough. Waves 5 to 22 ft.
110.0, June 1961, downstream.	−67.0	−3.50	Amplitude 25, length 600.	+14.00	Rough. Waves 5 to 25 ft.	Rough. Waves 3 to 6 ft.	Smooth.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, June 1961, upstream.	−32.0	.....	Amplitude 18, length 591.	.....	Smooth at start. Waves 2 to 8 ft. at top.	Smooth.	Rough. Waves 5 to 18 ft.
116.0, June 1961, downstream.	−34.0	+7.25	Amplitude 10, length 220.	+6.50	Rough. Waves 2 to 10 ft.	Irregular waves 2 to 3 ft.	Smooth.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, June 1961, upstream.	−50.0	.....	Amplitude 15, length 400.	.....	Rough. Waves 2 to 11 ft.	Rough. Waves 5 to 11 ft.	Rough. Waves 5 to 15 ft.
140.5, June 1961, Downstream.	−52.0	+4.75	Amplitude 16, length 612.	+10.00	Rough. Waves 2 to 12 ft.	Rough. Waves 4 to 12 ft.	Rough. Waves 10 to 16 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, June 1961, upstream.	−28.0	.....	Amplitude 12, length 294.	.....	Irregular waves 2 to 6 ft.	Irregular waves 1 to 3 ft.	Rough downstream. Waves 2 to 12 ft. Upstream ¾ is smooth.
154.0, June 1961, downstream.	−23.0	+8.75	Amplitude 11, length 283.	+5.00	Rough. Waves 5 to 8 ft.	Smooth.	Rough. Waves 4 to 11 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, June 1961, upstream.	−46.0	.....	Amplitude 11, length 98.	.....	Rough. Waves 2 to 9 ft.	Smooth.	Rough. Waves 3 to 11 ft.
163.0, June 1961, downstream.	−46.0	+ .25	Amplitude 13, length 303.	+3.50	Rough. Waves 4 to 13 ft.	Smooth.	Rough. Waves 6 to 10 ft.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, June 1961, upstream.	−47.0	.....	Amplitude 28, length 392.	.....	Rough. Waves 2 to 28 ft.	Irregular. Waves 2 to 4 ft.	Rough. Waves 8 to 20 ft.
167.0, June 1961, downstream.	−48.0	+7.25	Amplitude 20, length 404.	+12.50	Rough. Waves 6 to 18 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 8 to 20 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, June 1961, upstream.	−50.0	.....	Amplitude 17, length 554.	.....	Rough. Waves 3 to 17 ft.	Smooth.	Almost smooth. Waves 3 to 6 ft.
183.0, June 1961, downstream.	−49.0	−7.00	Amplitude 24, length 378.	+6.50	Rough. Waves 7 to 24 ft.	Rough. Waves 5 to 8 ft.	Rough. Waves 9 to 24 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, June 1961, upstream.	−37.0	.....	Amplitude 16, length 792.	.....	Rough. Waves 2 to 15 ft.	Smooth.	Rough. Waves 7 to 16 ft.
190.5, June 1961, downstream.	−41.0	−3.75	Amplitude 15, length 438.	+4.00	Irregular.	Irregular. Waves 2 to 3 ft.	Rough. Waves 7 to 15 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves	Top rough. Waves 6 to 10 ft.
197.5, June 1961, upstream.	−36.0	.....	Amplitude 17, length 594.	.....	Rough. Waves 2 to 11 ft.	Irregular. Waves 2 to 5 ft.	Rough. Waves 4 to 17 ft.
197.5, June 1961, downstream.	−37.0	+ .25	Amplitude 16, length 279.	+7.00	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 6 to 16 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, June 1961, upstream.	−42.0	.....	Amplitude 15, length 392.	.....	Rough. Waves 2 to 10 ft.	Irregular. Waves 1 to 3 ft.	Rough. Waves 5 to 15 ft.
204.0, June 1961, downstream.	−41.0	−7.75	Amplitude 16, length 286.	+4.50	Bottom rough. Upper part smooth.	Smooth and steep.	Rough. Waves 10 to 16 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, June 1961, upstream.	−40.0	.....	Amplitude 14, length 1,078.	.....	Rough. Waves 2 to 6 ft.	Irregular. Waves 2 to 5 ft.	Rough. Waves 8 to 14 ft.
212.0, June 1961, downstream.	−35.0	− .25	Amplitude 25, length 1,591	+5.00	Rough. Waves 2 to 6 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 15 to 25 ft.

See footnotes at end of table.

TABLE 6.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus June 1 to 6, 1961 (70° to 73° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, June 1961, upstream.	−33.0	.....	Amplitude 13, length 497.	.....	Rough. Waves 2 to 7 ft.	Smooth.	Rough. Waves 4 to 13 ft.
223.5, June 1961, downstream.	−32.0	−1.00	Amplitude 17, length 500.	+1.00	Rough. Waves 2 to 10 ft.	Smooth at top. Lower part has waves 3 to 5 ft.	Rough. Waves 7 to 17 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, June 1961, upstream.	−32.0	.....	Amplitude 21, length 306.	.....	Rough. Waves 3 to 21 ft.	Rough. Waves 3 to 19 ft. Lower half is smooth.	Rough. Waves 3 to 14 ft.
236.0, June 1961, downstream.	−41.0	−11.25	Amplitude 18, length 1,300.	+4.50	Rough. Waves 3 to 11 ft.	Smooth.	Rough. Waves 6 to 18 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, June 1961, upstream.	−33.0	.....	Amplitude 13, length 229.	.....	Rough. Waves 2 to 9 ft.	Smooth.	Rough. Waves 1 to 13 ft.
241.0, June 1961, downstream.	−38.0	−14.00	Amplitude 18, length 400.	+4.00	Rough. Waves 2 to 8 ft.	Smooth and steep.	Rough. Waves 9 to 18 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, June 1961, upstream.	−22.0	.....	Amplitude 15, length 300.		Rough. Waves 1 to 3 ft.	Smooth.	Upstream part rough. Waves 5 to 15 ft. Downstream part smooth.
251.0, June 1961, downstream.	−22.0	−7.50	Amplitude 15, length 444.	+1.00	Rough. Waves 1 to 6 ft.	Smooth.	Rough. Waves 5 to 15 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, June 1961, upstream.	−34.0	.....	Amplitude 16, length 1,060.	.....	Rough. Waves 2 to 10 ft.	Smooth.	Rough upstream. Waves 7 to 16 ft. Downstream part smooth.
258.5, June 1961, downstream.	−32.0	−14.00	Amplitude 16, length 355.	+1.00	Rough. Waves 1 to 3 ft.	Smooth.	Rough. Waves 6 to 17 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.

269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, June 1961, upstream.	−61.0	.....	Amplitude 12, length 200.	.....	Irregular. Waves 2 to 4 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 2 to 5 ft.
269.5, June 1961, downstream.	−59.0	−25.25	Amplitude 9, length 222.	−3.00	Rough. Waves 2 to 5 ft.	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 6 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, June 1961 upstream.	−28.0	.....	Amplitude 15, length 695.	.....	Rough. Waves 3 to 6 ft.	No system.	Rough. Waves 5 to 15 ft.
273.0, June 1961, downstream.	−23.0	−17.50	Amplitude 14, length 304.	−4.00	Rough. Waves 3 to 9 ft.	No system.	Rough. Waves 3 to 14 ft.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, June 1961, upstream.	−44.0	.....	Amplitude 20, length 556.	.....	Irregular. Waves 2 to 3 ft.	Smooth.	Rough. Waves 2 to 20 ft. Some smooth stretches.
279.0, June 1961 downstream.	−50.0	−16.25	Amplitude 18, length 665.	−4.00	Rough. Waves 3 to 16 ft.	Smooth.	Upstream half is rough. Waves 6 to 18 ft. Downstream half is smooth.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, June 1961, upstream.	−32.0	.....	Amplitude 15, length 953.	.....	Rough. Waves 3 to 6 ft.	Rough. Long waves, 5 to 15 ft.	Smooth.
286.5, June 1961, downstream.	−28.0	−23.50	Amplitude 17, length 380.	−9.00	Rough. Waves 2 to 9 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 6 to 17 ft.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, June 1961, upstream.	[3]	.....	Amplitude length	DID NOT RECORD POSITION ON UPSTREAM RUN.			
298.0, June 1961, downstream.	−21.0	.....	Amplitude 6, length 548.	.....	Irregular. Waves 1 to 3 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 2 to 5 ft.

[1] Above head of passes.
[2] Mean low water.
[3] No definite waves. Smooth undulations.

TABLE 7.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 16 to 19, 1962 (46° to 47° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upstream end rough. Waves 3 to 5 ft. Downstream part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Feb. 1962, upstream.	−78.8	.....	Amplitude 9, length 287.	.....	Smooth. General irregularities.	Smooth. General irregularities.	Smooth.
110.0, Feb. 1962, downstream.	−72.2	−14.00	Amplitude 7, length 119.	−1.50	Smooth to irregular.	Rough. Waves 2 to 6 ft.	Smooth.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upstream end. Then waves 2 to 3 ft.	Upstream end, waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Downstream end undulating. Upstream end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Feb. 1962, upstream.	−48.3	.....	Amplitude 13, length 287.	.....	Rough. Waves 1 to 7 ft. high.	Generally smooth.	Rough. Waves 5 to 13 ft. high.
116.0, Feb. 1962, downstream.	−39.6	−3.75	Amplitude 14, length 297.	+6.00	Rough. Waves 2 to 14 ft.	Irregular.	Rough. Waves 6 to 14 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upstream end rough. Waves 2 to 4 ft. Downstream end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end, waves 2 to 5 ft.	Upstream end rough. Waves 2 to 3 ft. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Feb. 1962, upstream.	−49.9	.....	Amplitude 11, length 191.	.....	Rough. Waves 7 ft. high.	Rough. Waves 2 to 6 ft. high.	Rough. Waves 3 to 7 ft. high.
140.5, Feb. 1962, downstream.	−54.1	+3.75	Amplitude 15, length 293.	+7.50	Rough. Waves 3 to 15 ft. high.	Rough. Waves 2 to 8 ft.	Top rough. Waves 5 to 15 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Feb. 1962, upstream.	−39.2	.....	Amplitude 7, length 223.	.....	Smooth, becoming rough with waves 2 to 5 ft.	Rough and irregular. Waves 2 to 7 ft.	Rough with waves 2 to 4 ft., close together.
154.0, Feb. 1962, downstream.	−38.7	−4.75	Amplitude 13, length 147.	+3.50	Rough. Waves 2 to 5 ft.	Rough. Waves 3 to 13 ft.	Rough. Waves 2 to 6 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth downstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Feb. 1962, upstream.	−56.1	.....	Amplitude 11, length 239.	.....	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 6 ft.	Rough with larger waves, 5 to 11 ft.
163.0, Feb. 1962, downstream.	−41.7	−2.65	Amplitude 10, length 234.	+2.00	Rough. Waves 2 to 7 ft.	Irregular. Waves 2 to 4 ft.	Smooth and steep.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants of large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Feb. 1962, upstream.	−52.7	.....	Amplitude 20, length 447.	.....	Rough. Starting out with small waves becoming larger at top of crossing.	Rough. No small waves. Large waves 10 to 16 ft.	Rough. Waves 10 to 20 ft.
167.0, Feb. 1962, downstream.	−54.2	+1.30	Amplitude 22, length 733.	+9.50	Rough. Waves 3 to 12 ft.	Rough. Waves 3 to 10 ft.	Rough. Waves 6 to 22 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Feb. 1962, upstream.	−45.8	.....	Amplitude 13, length 447.	.....	Fairly regular. Waves 2 to 3 ft.	Smooth.	Large waves 5 to 13 ft.
183.0, Feb. 1962, downstream.	−40.4	− .60	Amplitude 15, length 510.	+ .00	Smooth.	Smooth.	Rough. Large waves 8 to 15 ft. Lower half smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Feb. 1962, upstream.	−42.0	.....	Amplitude 12, length 255.	.....	Rough. Waves 2 to 7 ft.	Rough. Waves 3 to 11 ft.	Rough. Waves 6 to 12 ft.
190.5, Feb. 1962, downstream.	−46.6	−9.05	Amplitude 15, length 210.	+2.00	Rough. Waves 3 to 13 ft.	Rough. Waves 3 to 13 ft.	Rough. Waves 5 to 15 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Feb. 1962, upstream.	−40.2	.....	Amplitude 16, length 187.	.....	Regular at bottom. Upper part waves 2 to 9 ft.	Rough .Waves 3 to 16 ft.	Waves 2 to 7 ft.
197.5, Feb. 1962, downstream.	−39.8	−3.25	Amplitude 14, length 150.	+5.50	Rough. Waves 3 to 14 ft.	Rough. Waves 3 to 12 ft.	Rough. Waves 5 to 11 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Feb. 1962, upstream.	−41.5	.....	Amplitude 14, length 249.	.....	Rough. Waves 6 to 11 ft.	Regular, smooth, no wave system.	Rough. Waves 3 to 14 ft.
204.0, Feb. 1962, downstream.	−39.2	−6.60	Amplitude 17, length 270.	+4.50	Rough. Waves 2 to 17 ft.	Generally smooth, some waves 2 to 5 ft.	Irregular. Waves 2 to 5 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 6 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.

See footnotes at end of table.

TABLE 7.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 293 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 16 to 19, 1962 (46° to 47° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 7 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Feb. 1962, upstream.	−40.4	.....	Amplitude 13, length 280.	.....	Irregular small waves becoming fewer and larger nearing top.	Rough. Waves 3 to 8 ft.	Top rough. Large waves 6 to 13 ft.
212.0, Feb. 1962, downstream.	−41.3	−3.60	Amplitude 14, length 450.	−1.00	Rough. Waves 2 to 12 ft.	Irregular. Waves 2 to 4 ft.	Rough. Waves 7 to 14 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Feb. 1962, upstream.	−27.0	.....	Amplitude 16, length 498.	.....	Rough. Waves 2 to 9 ft.	Regular. No wave system.	Rough. Waves 2 to 7 ft.
223.5, Feb. 1962, downstream.	−30.8	+2.60	Amplitude 17, length 602.	+2.50	Rough. Waves 7 to 17 ft.	Rough. Waves 6 to 17 ft.	Irregular. Waves 2 to 10 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth. Waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Feb. 1962, upstream.	−35.9	.....	Amplitude 16, length 311.	.....	Rough. Waves 3 to 10 ft.	Rough. Larger waves 6 to 10 ft.	Rough. Waves 3 to 8 ft.
236.0, Feb. 1962, downstream.	−30.6	−8.00	Amplitude 16, length 301.	+1.00	Rough. Waves 2 to 5 ft.	Smooth.	Rough. Waves 2 to 16 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upstream part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upstream part, with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Feb. 1962, upstream.	−30.7	.....	Amplitude 8, length 222.	.....	Irregular waves 2 to 8 ft.	Smooth.	Smooth.
241.0, Feb. 1962, downstream.	−31.0	−9.40	Amplitude 7, length 211.	−4.00	Rough. Waves 2 to 6 ft.	Smooth.	Rough. Waves 2 to 4 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Feb. 1962, upstream.	.....	.....	Amplitude length	.....	WENT THROUGH PROFIT ISLAND CHUTE INSTEAD OF RIVER.		
251.0, Feb. 1962, downstream.	−28.1	.....	Amplitude 21, length 421.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves to 21 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.

258.5, Feb. 1962, upstream.	.....	.....	Amplitude length	.....	AT MOUTH OF PROFIT ISLAND CHUTE. DID NOT GET CROSSING.		
258.5, Feb. 1962, downstream.	−36.5	.....	Amplitude 16, length 421.	.....	Smooth.	Irregular.	Rough. Waves 4 to 16 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough. Waves 3 to 10 ft. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough, with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Feb. 1962, upstream.	−39.3	.....	Amplitude 18, length 634.	.....	Rough. Waves 5 to 10 ft.	Generally smooth.	No real "top", a single large wave.
269.5, Feb. 1962, downstream.	−43.5	−6.65	Amplitude 17, length 332.	+4.00	Rough. Waves 3 to 10 ft.	Smooth.	Rough. Waves 5 to 17 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000	.....	Regular with 2 to 5 ft. long undulations.	Smooth, with long undulations.	Rough upstream end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Feb. 1962, upstream.	− 8.3	.....	Amplitude 13, length 507.	.....	Irregular. Waves 2 to 3 ft.	Smooth at top. Irregular at bottom.	Smooth.
273.0, Feb. 1962, downstream.	−19.1	−5.70	Amplitude 12, length 277.	−6.00	Rough. Waves 6 to 12 ft.	Irregular. Waves 2 to 4 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Feb. 1962, upstream.	−24.8	.....	Amplitude 23, length 1,110.	.....	Irregular. Waves 2 to 9 ft.	Smooth.	Rough. Large waves 6 to 23 ft.
279.0, Feb. 1962, downstream.	−32.0	+2.35	Amplitude 18, length 970.	−2.50	Rough. Waves 3 to 12 ft.	Irregular.	Rough. Waves 10 to 18 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Feb. 1962, upstream.	−28.7	.....	Amplitude 6, length 349.	.....	Irregular. Waves 2 to 6 ft.	Smooth.	Very smooth.
286.5, Feb. 1962, downstream.	−24.0	−19.85	Amplitude 5, length 208.	−19.50	Rough. Waves 2 to 5 ft.	Smooth.	Very smooth.
298.0, Aug. 1958, upstream.	+ 0.5	.....	Amplitude 10, length 450.	.....	Long undulations, 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Feb. 1958, upstream.	−11.0	.....	Amplitude 8, length 153.	.....	Rough. Waves 2 to 3 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 2 to 3 ft.
298.0, Feb. 1962, downstream.	−4.5	−6.75	Amplitude 3, length 69.	−5.00	Same.	Irregular.	Rough. Waves 2 to 3 ft.

[1] Above head of passes.
[2] Mean low water.

TABLE 8.—*Summary of data of effect of water temperature on riverbed configuration, Mississippi River, mile 103 to 298 A.H.P., August 4 to 18, 1958 (82° to 83° F.), versus temperatures of other thalweg runs*

Crossing mileage A.H.P.	Thalweg run date	Temp. range (°F.)	High point M.L.W.[1] (feet)	Difference in elevation versus 1958 (feet)	Wave amplitude and length (feet)	Difference in amplitude versus 1958 (feet)	Crossing mileage A.H.P.	Thalweg run dates	Temp. range (°F.)	High point M.L.W.[1]	Difference in elevation versus 1958 (feet)	Wave amplitude and length[1] (feet)	Difference in amplitude versus 1958 (feet)
110.0	Aug. 4–18, 1958	82–83	−61.5	.....	Amplitude, 10, length, 170.	.....		May 8–12, 1961	64–66	−32.0	+ 3.25	Amplitude, 14, length, 400.	+ 2.00
	June 1–6, 1961	70–73	−67.0	− 3.50	Amplitude, 23, length, 497.	+14.00		Apr. 21–27, 1960	56–61	−42.5	− 7.50	Amplitude, 12, length, 450.	+ .50
	May 8–12, 1961	64–66	−71.0	− 9.50	Amplitude, 11, length, 200.	+ 1.50		Mar. 20–25, 1961	55–58	−42.5	− 7.25	Amplitude, 17, length, 400.	+ 5.00
	Apr. 21–27, 1960	56–61	−64.5	− 3.00	Amplitude, 7, length, 220.	− 2.00		Feb. 16–19, 1962	46–47	−44.3	− 9.05	Amplitude, 13, length, 233.	+ 2.00
	Mar. 20–25, 1961	55–58	−70.5	− 9.00	[2]	.....		Feb. 9–18, 1959	42–43	−37.8	− 2.50	Amplitude, 7, length, 140.	− 4.50
	Feb. 16–19, 1962	46–47	−75.0	−14.00	Amplitude, 8, length, 203.	− 1.50							
	Feb. 9–18, 1959	42–43	−67.0	− 5.50	Amplitude, 6, length, 190.	− 5.50	197.5	Aug. 4–18, 1958	82–83	−36.8	.....	Amplitude, 10, length, 175.	.....
								June 1–6, 1961	70–73	−36.5	+ 0.25	Amplitude, 16, length, 437.	+ 7.00
116.0	Aug. 4–18, 1958	82–83	−40.3	.....	Amplitude, 8, length, 145.	.....		May 8–12, 1961	64–66	−29.5	+ 7.25	Amplitude, 20, length, 925.	+10.50
	June 1–6, 1961	70–73	−33.0	+ 7.25	Amplitude, 14, length, 406.	+ 6.50		Apr. 21–27, 1960	56–61	−36.8	+ .00	Amplitude, 7, length, 350.	− 3.00
	May 8–12, 1961	64–66	−36.0	+ 4.25	Amplitude, 13, length, 300.	+ 6.00		Mar. 20–25, 1961	55–58	−41.5	− 4.75	Amplitude, 9, length, 275.	− .50
	Apr. 21–27, 1960	56–61	−41.3	− 1.00	Amplitude, 7, length, 125.	− .50		Feb. 16–19, 1962	46–47	−40.0	− 3.25	Amplitude, 15, length, 169.	+ 5.50
	Mar. 20–25, 1961	55–58	−49.0	+ 8.75	Amplitude, 8, length, 200,	+ .50		Feb. 9–18, 1959	42–43	−38.5	− 1.75	Amplitude, 7, length, 175,	− 2.00
	Feb. 16–19, 1962	46–47	−44.0	+ 3.75	Amplitude, 13, length, 292.	+ 6.00							−
	Feb. 9–18, 1959	42–43	−44.5	− 4.25	Amplitude, 3, length, 53.	− 4.50	204.0	Aug. 4–18, 1958	82–83	−33.7	.....	Amplitude, 11, length, 180.	.....
								June 1–6, 1961	70–73	−41.5	− 7.75	Amplitude, 15, length, 339,	+ 4.50
140.5	Aug. 4–18, 1958	82–83	−55.8	.....	Amplitude, 5, length 130.	.....		May 8–12, 1961	64–66	−32.0	+ 1.75	Amplitude, 14, length, 650.	+ 3.00
	June 1–6, 1961	70–73	−51.0	+ 4.75	Amplitude, 15, length, 506.	+10.00		Apr. 21–27, 1960	56–61	−38.8	− 5.00	Amplitude, 13, length, 325.	+ 2.00
	May 8–12, 1961	64–66	−48.5	+ 7.25	Amplitude, 14, length, 400.	+ 8.00		Mar. 20–25, 1961	55–58	−47.0	−13.25	Amplitude, 8, length, 250.	− 3.50
	Apr. 21–27, 1960	56–61	−51.8	+ 4.00	Amplitude, 10, length, 195.	+ 4.50		Feb. 16–19, 1962	46–47	−40.4	− 6.60	Amplitude, 15, length, 260.	+ 4.50
	Mar. 20–25, 1961	55–58	−53.0	+ 2.75	Amplitude, 10, length, 325.	+ 4.50		Feb. 9–18, 1959	42–43	−38.8	− 5.00	Amplitude, 7, length, 135.	− 3.50
	Feb. 16–19, 1962	46–47	−52.0	+ 3.75	Amplitude, 13, length, 242.	+ 7.50	212.0	Aug. 4–18, 1958	82–83	−37.3	.....	Amplitude, 15, length, 225.	.....
	Feb. 9–18, 1959	42–43	−58.5	− 2.75	[3]	− 5.50		June 1–6, 1961	70–73	−37.5	− 0.25	Amplitude, 20, length, 1335.	+ 5.00
154.0	Aug. 4–18, 1958	82–83	−34.3	.....	Amplitude 6, length, 128.	.....		May 8–12, 1961	64–66	−28.0	+ 9.25	Amplitude, 23, length, 1050.	+ 9.00
	June 1–6, 1961	70–73	−26.0	+ 8.75	Amplitude, 11, length, 289.	+ 5.00		Apr. 21–27, 1960	56–61	−48.0	−10.75	Amplitude, 10, length, 225,	− 5.00
	May 8–12, 1961	64–66	−26.0	+ 8.25	Amplitude, 13, length, 775.	+ 6.00		Mar. 20–25, 1961	55–58	−42.0	− 4.75	Amplitude, 9, length, 200.	− 5.50

	Apr. 21–27, 1960.	56–61	−36.3	− 2.00	Amplitude, 9, length, 190.	+ 2.00		Feb. 16–19, 1962.	46–47	−40.9	− 3.60	Amplitude, 14, length, 365.	− 1.00
	Mar. 20–25, 1961.	55–58	−37.5	− 3.25	Amplitude, 15, length, 163.	+ 8.50		Feb. 9–18, 1959..	42–43	−41.5	− 4.25	Amplitude, 9, length, 145,	− 5.50
	Feb. 16–19, 1962.	46–47	−39.0	− 4.75	Amplitude, 10, length, 185.	+ 3.50	223.5	Aug. 4–18, 1958..	82–83	−31.5	.....	Amplitude, 13, length, 463,	.....
	Feb. 9–18, 1959..	42–43	−35.8	− 1.50	Amplitude, 4, length, 88.	− 3.00		June 1–6, 1961...	70–73	−32.5	− 1.00	Amplitude, 15, length, 498.	+ 1.00
163.0	Aug. 4–18, 1958..	82–83	−46.3	.....	Amplitude, 8, length, 130.	.....		May 8–12, 1961..	64–66	−31.3	+ .25	Amplitude, 15, length, 600.	+ 1.00
	June 1–6, 1961...	70–73	−46.0	+ 0.25	Amplitude, 12, length, 201.	+ 3.50		Apr. 21–27, 1960.	56–61	−29.0	+ 2.50	Amplitude, 9, length, 260.	− 5.50
	May 8–12, 1961..	64–66	−42.5	+ 3.75	Amplitude, 8, length, 275.	− .50		Mar. 20–25, 1961.	55–58	−52.0	−20.50	Amplitude, 14, length, 475.	+ .50
	Apr. 21–27, 1960.	56–61	−46.5	− .25	Amplitude, 8, length, 155.	− 1.00		Feb. 16–19, 1962.	46–47	−28.9	+ 2.60	Amplitude, 16, length, 550.	+ 2.50
	Mar. 20–25, 1961.	55–58	−50.0	− 3.75	Amplitude, 14, length, 200.	+ 5.00		Feb. 9–18, 1959.	42–43	−33.5	− 2.00	Amplitude, 13, length, 500.	− 1.00
	Feb. 16–19, 1962.	46–47	−48.9	− 2.65	Amplitude, 10, length, 237.	+ 2.00	236.0	Aug. 4–18, 1958..	82–83	−25.3	.....	Amplitude, 15, length, 225.	.....
	Feb. 9–18, 1959..	42–43	−48.5	− 2.00	Amplitude, 3, length, 93.	− 5.5		June 1–6, 1961...	70–73	−37.0	−11.25	Amplitude, 20, length, 803.	+ 4.50
167.0	Aug. 4–18, 1958..	82–83	−54.8	.....	Amplitude, 12, length, 235.	.....		May 8–12, 1961..	64–66	−30.0	− 4.75	Amplitude, 12, length, 375.	− 3.00
	June 1–6, 1961...	70–73	−47.0	+ 7.25	Amplitude, 24, length, 398.	+12.50		Apr. 21–27, 1960.	56–61	−34.0	− 8.75	Amplitude, 23, length, 1175.	+ 8.00
	May 8–12, 1961..	64–66	−47.0	+ 7.75	Amplitude, 16, length, 500.	+ 4.50		Mar. 20–25, 1961.	55–58	−46.5	−21.25	Amplitude, 4, length, 150.	−10.50
	Apr. 21–27, 1960.	56–61	−58.8	− 4.00	Amplitude, 10, length, 285.	− 1.50		Feb. 16–19, 1962.	46–47	−33.3	− 8.00	Amplitude, 16, length, 306.	+ 1.00
	Mar. 20–25, 1961.	55–58	−52.5	+ 2.25	Amplitude, 11, length, 335.	− .50		Feb. 9–18, 1959..	42–43	−30.8	− 5.50	Amplitude, 13, length, 215,	− 2.00
	Feb. 16–19, 1962.	46–47	−53.5	+ 1.30	Amplitude, 21, length, 590.	+ 9.50	241.0	Aug. 4–18, 1958..	82–83	−21.5	.....	Amplitude, 12, length, 165.	.....
	Feb. 9–18, 1959..	42–43	−54.3	+ .50	Amplitude, 9, length, 200.	− 2.50		June 1–6, 1961...	70–73	−36.0	−14.00	Amplitude, 15, length, 315.	+ 4.00
183.0	Aug. 4–18, 1958..	82–83	−42.5	.....	Amplitude, 14, length, 293.	.....		May 8–12, 1961..	64–66	−35.0	−13.50	Amplitude, 9, length, 175.	− 2.50
	June 1–6, 1961...	70–73	−49.5	− 7.00	Amplitude, 21, length, 466.	+ 6.50		Apr. 21–27, 1960.	56–61	−26.0	− 4.50	[3]	−12.00
	May 8–12, 1961..	64–66	−45.8	− 3.25	Amplitude, 20, length, 700.	+ 6.00		Mar. 20–25, 1961.	55–58	−36.0	−14.50	Amplitude, 10, length, 138.	− 1.50
	Apr. 21–27, 1960.	56–61	−46.3	− 3.75	Amplitude, 15, length, 550.	+ 1.00		Feb. 16–19, 1962.	46–47	−30.9	− 9.40	Amplitude, 8, length, 217.	− 4.00
	Mar. 20–25, 1961.	55–58	−43.5	− 1.00	Amplitude, 13, length, 173.	− 1.00		Feb. 9–18, 1959..	42–43	−24.5	− 3.00	Amplitude, 12, length, 325.	− .50
	Feb. 16–19, 1962.	46–47	−43.1	− .60	Amplitude, 14, length, 479.	+ .00	251.0	Aug. 4–18, 1958..	82–83	−14.5	.....	Amplitude, 14, length, 263.	.....
	Feb. 9–18, 1959..	42–43	−43.9	− 1.25	Amplitude, 13, length, 215,	− 1.50		June 1–6, 1961...	70–73	−22.0	− 7.50	Amplitude, 15, length, 372.	+ 1.00
190.5	Aug. 4–18, 1958..	82–83	−35.3	.....	Amplitude, 12, length, 170.	.....		May 8–12, 1961..	64–66	−13.5	+ 1.00	Amplitude, 12, length, 325.	− 2.50
	June 1–6, 1961...	70–73	−39.0	− 3.75	Amplitude, 15, length, 615.	+ 4.00		Apr. 21–27, 1960.	56–61	−20.3	− 5.75	Amplitude, 12, length, 275.	− 2.00

See footnotes at end of table.

TABLE 8.—*Summary of data of effect of water temperature on riverbed configuration, Mississippi River, mile 103 to 298 A.H.P., August 4 to 18, 1958 (82° to 83° F.), versus temperatures of other thalweg runs—Continued*

Crossing mileage A.H.P.	Thalweg run dates	Temp. range (°F.)	High point M.L.W.[1]	Difference in elevation versus 1958 (feet)	Wave amplitude and length (feet)	Difference in amplitude versus 1958 (feet)
251.0	Mar. 20–25, 1961	55–58	−31.3	−16.75	Amplitude, 14, length, 213.	+ .00
	Feb. 16–19, 1962	46–47	([4])	([4])	([4])	([4])
	Feb. 9–18, 1959	42–43	−19.8	− 5.25	Amplitude, 9, length, 125.	− 4.50
258.5	Aug. 4–18, 1958	82–83	−19.0	.....	Amplitude, 15, length, 500.	.....
	June 1–6, 1961	70–73	−33.0	−14.00	Amplitude, 16, length, 708.	+ 1.00
	May 8–12, 1961	64–66	−31.5	−12.00	Amplitude, 14, length, 250.	− 1.50
	Apr. 21–27, 1960	56–61	−28.0	− 9.00	Smooth	−15.50
	Mar. 20–25, 1961	55–58	−26.9	− 7.75	Amplitude, 17, length, 350.	− 1.50
	Feb. 16–19, 1962	46–47	([4])	([4])	([4])	([4])
	Feb. 9–18, 1959	42–43	−21.3	− 1.25	Amplitude, 11, length, 215,	− 4.00
269.5	Aug. 4–18, 1958	82–83	−35.0	.....	Amplitude, 13, length, 238.	.....
	June 1–6, 1961	70–73	−60.0	−25.25	Amplitude, 11, length, 211.	− 3.00
	May 8–12, 1961	64–66	−52.0	−17.25	Amplitude, 16, length, 225.	+ 2.00
	Apr. 21–27, 1960	56–61	−41.8	− 7.00	Amplitude, 7, length, 163.	− 6.50
	Mar. 20–25, 1961	55–58	−47.8	−13.00	Amplitude, 12, length, 288.	− 1.50
	Feb. 16–19, 1962	46–47	−41.4	− 6.65	Amplitude, 17, length, 483.	+ 4.00
	Feb. 9–18, 1959	42–43	−35.5	− .75	Amplitude, 11, length, 250.	− 2.50
273.0	Aug. 4–18, 1958	82–83	− 8.0	.....	Amplitude, 19, length, 1125.	.....
	June 1–6, 1961	70–73	−25.5	−17.50	Amplitude, 14, length, 500.	− 4.00
	May 8–12, 1961	64–66	−17.0	− 8.50	Amplitude, 14, length, 425.	− 4.50
	Apr. 21–27, 1960	56–61	−31.8	− 5.75	([5])	−18.50
	Mar. 20–25, 1961	55–58	−27.8	−19.75	Amplitude, 11, length, 250.	− 7.50
	Feb. 16–19, 1962	46–47	−13.7	− 5.70	Amplitude, 13, length, 392.	− 6.00
279.0	Aug. 4–18, 1958	82–83	−30.8	.....	Amplitude, 22, length, 350.	.....
	June 1–6, 1961	70–73	−47.0	−16.25	Amplitude, 19, length, 611.	− 4.00
	May 8–12, 1961	64–66	−32.3	− 1.50	Amplitude, 20, length, 450.	− 2.50
	Apr. 21–27, 1960	56–61	−24.3	+ 6.50	Amplitude, 8, length, 470.	−14.50
	Mar. 20–25, 1961	55–58	−37.0	− 6.25	Amplitude, 15, length, 475.	− 7.50
	Feb. 16–19, 1962	46–47	−28.4	+ 2.35	Amplitude, 21, length, 1040.	− 2.50
	Feb. 9–18, 1959	42–43	−30.0	− 1.00	Amplitude, 8, length, 175.	− 14.5
286.5	Aug. 4–18, 1958	82–83	− 6.5	.....	Amplitude, 25, length, 388.	.....
	June 1–6, 1961	70–73	−30.0	−23.50	Amplitude, 16, length, 667.	− 9.00
	May 8–12, 1961	64–66	−22.5	−16.00	Amplitude, 15, length, 900.	−10.00
	Apr. 21–27, 1960	56–61	− 9.0	− 2.50	Amplitude, 20, length, 2200.	− 5.00
	Mar. 20–25, 1961	55–58	−30.5	−24.0	Amplitude, 7, length, 150.	−18.00
	Feb. 16–19, 1962	46–47	−26.4	−19.85	Amplitude, 5, length, 279.	−19.50
	Feb. 9–18, 1959	42–43	−23.8	−17.25	Amplitude, 9, length, 305.	−16.00
298.0	Aug. 4–18, 1958	82–82	− 1.0	.....	Amplitude, 10, length, 350.	.....
	June 1–6, 1961	70–73	−21.0	.....	Amplitude, 6, length, 548.	.....
	May 8–12, 1961	64–66	−16.5	−15.50	([6])	.....
	Apr. 21–27, 1960	56–61	− 8.8	− 7.75	Amplitude, 9, length, 400.	− 1.00
	Mar. 20–25, 1961	55–58	−21.5	−20.50	Amplitude, 10, length, 138.	− 1.00
	Feb. 16–19, 1962	46–47	− 7.8	− 6.75	Amplitude, 6, length, 111.	− 5.00
	Feb. 9–18, 1959	42–43	− 7.5	− 6.50	Amplitude, 9, length, 313.	− 1.50

[1] Average of upstream and downstream runs for same river rise.
[2] No wave system.
[3] Smooth.
[4] No data—boat went through Profit Island Chute instead of river.
[5] Smooth—no waves.
[6] No wave pattern.

TABLE 9.—*Mississippi River stage, discharge, and temperature data arranged to demonstrate effect of temperature on discharge for the period involved in thalweg profile runs, August 1958 through February 1962, at Red River Landing*[1]

Item [2]	Red River landing gage stages														
	32.0	32.5	33.0	33.5	34.0	34.5	35.0	35.5	36.0	36.5	37.0	37.5	38.0	8.5	39.0
Date	3/7/61	3/8/61	3/9/61	3/10/61	3/11/61	3/12/61	/313/61	3/14/61	3/15/61	3/16/61	3/16/61	3/17/61	3/17/61	3/18/61	3/19/61
Temp. °F.	54	54	53	54	54	53	53	54	54	54	54	54	54	58	58
Discharge c.f.s.	726	743	756	769	780	789	799	808	820	833	847	862	879	898	917
Date	2/23/59	2/23/59	2/25/59	4/11/60	3/8/59	3/1/59	4/14/60	4/15/60	3/5/62	3/6/62	3/6/62	3/7/62	2/8/62	2/10/62	3/9/62
Temp. °F.	46	46	46	60	54	51	54	54	48	46	46	46	44	44	46
Discharge c.f.s.	681	688	690	713	725	744	755	768	779	797	814	830	850	864	882
Date	4/9/60	4/9/60	4/10/60	3/9/59	4/12/60	4/14/60	2/1/62	2/2/62	2/3/62	2/4/62	4/22/60	2/6/62	3/8/62	3/9/62	5/3/61
Temp. °F.	54	54	55	54	55	54	40	41	42	42	59	42	46	46	63
Discharge c.f.s.	643	666	688	706	725	742	752	765	778	792	809	825	847	863	820
Date	3/14/59	3/12/59	3/10/59	2/28/59	1/31/62	2/28/59	3/4/62	3/4/62	4/18/60	4/19/60	2/4/62	4/23/60	2/16/62	5/7/61	5/11/61
Temp. °F.	55	57	56	46	40	46	49	49	56	57	42	59	46	65	65
Discharge c.f.s.	641	651	681	692	723	738	738	760	771	778	805	817	804	795	815
Date	7/26/58	7/27/58	3/1/62	3/2/62	2/24/62	1/31/62	2/21/62	2/20/62	2/19/62	2/19/62	2/18/62	2/17/62	6/12/61	5/9/61	6/12/61
Temp. °F.	84	84	50	51	48	40	47	47	47	47	46	46	75	64	75
Discharge c.f.s.	616	622	650	670	705	737	723	739	756	768	781	792	698	793	722
Date	7/24/58	8/23/58	2/25/62	2/24/62	3/3/62	3/3/62	7/30/58	5/2/60	8/2/58	8/16/58	4/29/60	4/27/60		6/10/61	
Temp. °F.	84	83	48	48	49	49	85	66	84	84	64	62		74	
Discharge c.f.s.	580	595	649	668	690	712	668	686	718	738	765	786		711	
Date	5/5/60	5/5/60	7/27/58	7/28/58	2/23/62	2/22/62	8/21/58	8/1/58	6/1/60	4/30/60	6/13/61	6/12/61			
Temp. °F.	66	66	84	84	47	47	83	84	80	64	75	75			
Discharge c.f.s.	565	558	629	637	688	706	666	683	713	740	671	685			
Date	6/17/61	6/16/61	8/23/58	8/22/58	7/29/58	7/30/58	5/2/60	8/7/58	8/2/58	8/3/58					
Temp. °F.	76	75	83	83	84	85	66	84	84	84					
Discharge c.f.s.	545	558	612	625	647	657	665	728	701	725					
Date			5/4/60	5/4/60	8/22/58	8/21/58	6/14/61	6/14/61		6/13/61					
Temp. °F.			66	66	83	83	75	75		75					
Discharge c.f.s.			600	617	639	653	613	628		657					
Date			6/16/61	6/15/61	5/4/60	5/4/60									
Temp. °F.			75	75	66	67									
Discharge c.f.s.			563	574	633	649									
Date					6/15/61	6/14/61									
Temp. °F.					75	75									
Discharge c.f.s.					586	599									

[1] Except for February 1959 run, when later rise in March was used.
[2] Discharges are in thousands cubic feet per second from published Mississippi River Commission records.

feet) as the other runs. A glance at the actual values in the tables suggests that strong confirmation for the thesis of this paper could have been supplied. It is because of this reasoning that the results of the February 1959 run were not discarded: It was felt they serve a limited purpose if their limitations are kept in mind.

(2) Table 8: a summary tabulation of the major riverbed data presented in the tables above and arranged in order of temperature.

The most important riverbed statistics are considered to be the height referred to mean low water (M.L.W.) of the highest point of each of the crossings and the amplitude of the largest sand wave on each of the crossings. The tabulations show a general lowering of crossings with reduced water temperature and a general reduction in amplitude of the largest sand waves: Both of these statistics vary from crossing to crossing for each of the comparisons. Also, in a few cases, the crossings were not lower for the colder water. Finally, the lowering of crossings was more consistent than the reduction in amplitude of the maximum sand waves. The sand wave changes are complicated by the fact that, up to a certain stage (near bank full), sand waves become larger (see fig. 1); after which they tend to smooth out,[5] until possibly another series of (perhaps basically different type) waves or dunes tends to form again at high flood stages.

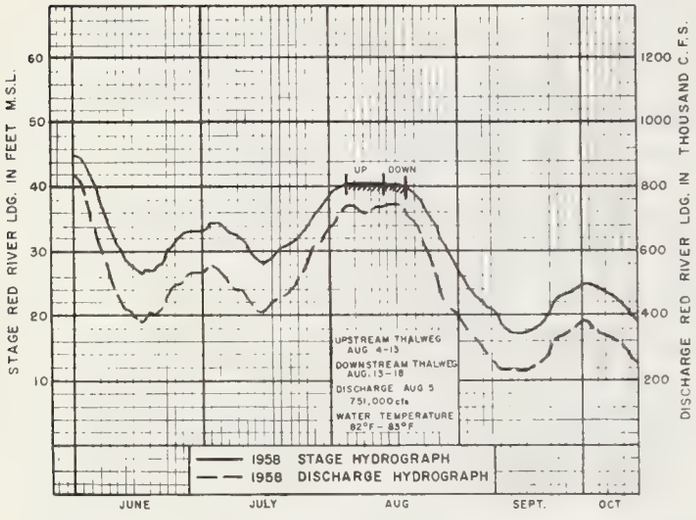

FIGURE 3. — 1958 stage and discharge hydrographs, Mississippi River, Red River Landing, La.

It is suggested that these deviations from a general trend may also, to some extent, be

[5] In which respect, the river conforms with Vanoni and Brooks' experience in their flume studies (see footnote 8) where they found that increased flow caused the ripples or dunes to disappear and the model bed to become smooth.

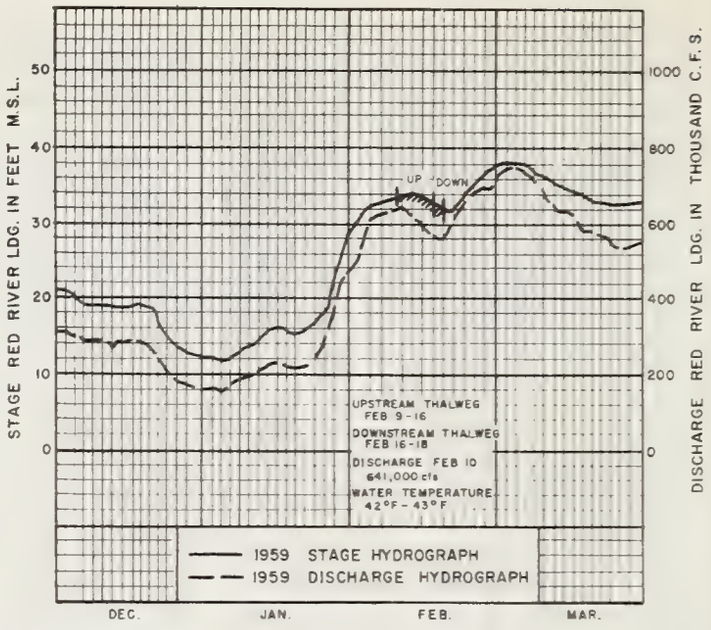

FIGURE 4. — 1959 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

indications that we are dealing with a natural river flowing in a channel carved through somewhat heterogeneous formations containing important anomalies. That is to say, if the river flowed through homogeneous formations, with no anomalies, it would have a uniform alinement, like the channels in the Tyler-Friedkin models, and every crossing would behave in exactly the same way under the influence of varying temperature. Also, the comparisons as given are based upon crossings only. This is done for two reasons: First, it is believed that

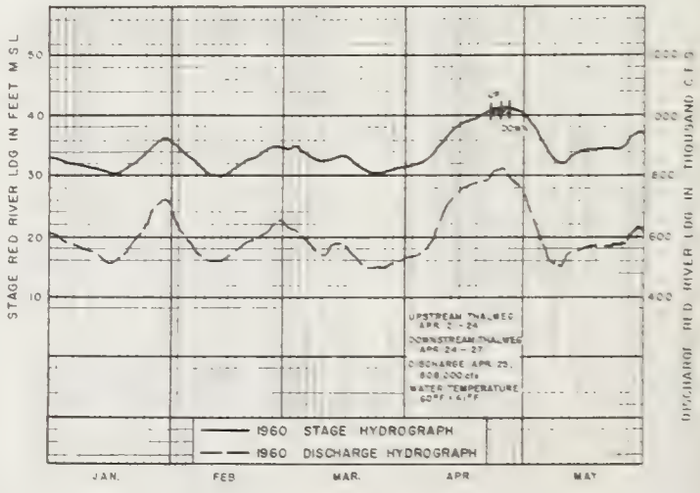

FIGURE 5. — 1960 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

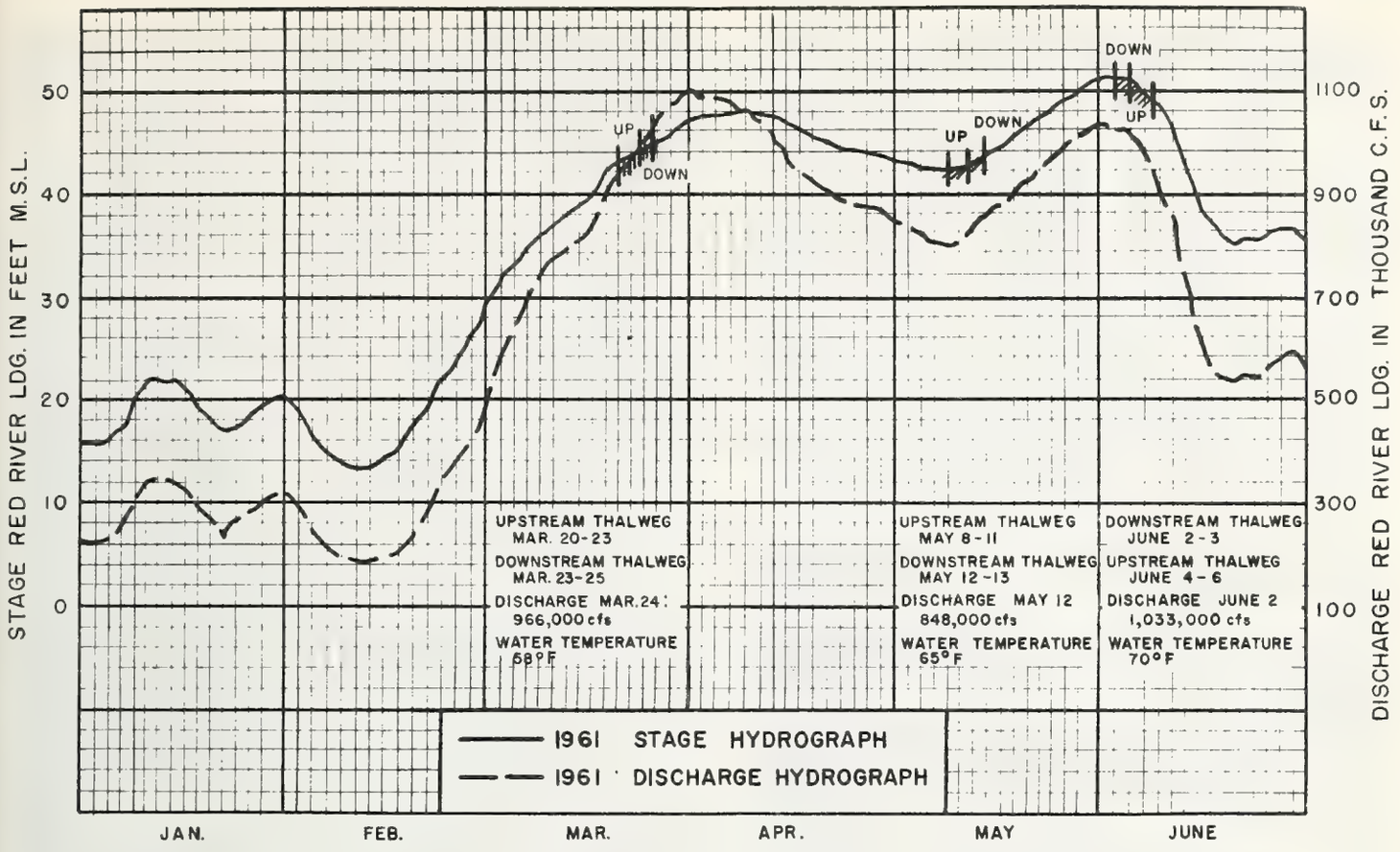

FIGURE 6. — 1961 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

the crossings, being submerged weirs in effect, have the greatest influence on flow; and, second, it is more difficult to control the lateral position of the survey boat when running around bends. It may be mentioned that the basic data (thalweg profiles) indicate that the bends also deepen under the effect of colder water.

As already indicated, another basis for the thesis of this paper is afforded by a special presentation, based upon certain official published discharges of the Mississippi River Commission at Red River Landing (mile 300.6 A.H.P.)[6] at an approximate stage of 37 feet. Table 9 presents these data. Study of this table indicates that failure to observe temperature in discharge observations and to make proper allowance for its effect on flow in alluvial rivers may introduce an element of error into discharge measurements and computations.

[6] Just below Old River, after the Mississippi has divided; one part going down the Atchafalaya and the other part going down the Mississippi past New Orleans to the Gulf.

[7] This effect is confined to alluvial rivers; that is, there is no reason to expect any similar effect on the flow in canals, pipes, etc.

[8] See U.S. Army Engineer Division, Missouri River, Report E-68, Dec. 1957, "Laboratory Studies of Roughness and Suspended Load of Alluvial Streams," by Vito A. Vanoni and Norman H. Brooks.

Mechanisms by Which Water Temperature Affects Bed Configuration

Throughout the field of civil engineering, the primary effect of water temperature is ignored; that is to say, the change in viscosity of water in the normal range of temperatures has such a slight effect upon the manner in which water behaves that it can be, and is, ignored.

This fact undoubtedly is the reason for the failure to explore the possible secondary effect of temperature on flow in alluvial rivers;[7] that is, the effect of viscosity on the "fall velocity" of a particle of sediment and all the subsidiary phenomena that stem from this basic phenomenon. This "fall velocity" essentially determines or controls the ability of the water to put into suspension or motion certain sizes and quantities of sediment and ultimately, therefore, everything having to do with systematic variations in riverbed configuration.

Working with this major factor that affects riverbed configuration is another recently discovered factor — that muddy water has less resistance to flow than clear water and the muddier the water (within reason) the less the resistance.[8] Vanoni and Brooks' explanation, it may be noted, is that sediment particles dampen

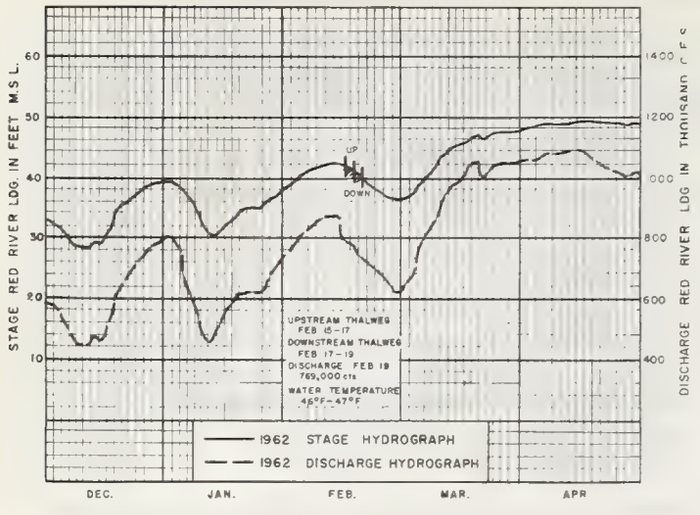

FIGURE 7. — 1962 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

turbulence, hence the more particles of sediment the less turbulence, hence the less resistance to flow.

The present discovery and the Vanoni-Brooks discovery work together to account for the overall effect of temperature on riverbed configuration and, in turn, the effect on resistance to flow.

Conclusions

The author in recent rereading of various papers dealing with flow in rivers, movement of sediment and related subjects, notes that several writers were moving in the direction of recognizing the broad effect of temperature: In each instance, however, they veered off course just about the time when recognition might have been made. These other observers, however, did not have the fathometer at their disposal as a ready means of obtaining the many miles of thalweg profiles necessary for an empirical study of this phenomenon. Nevertheless and despite the foregoing, the existence of this hiatus in river hydraulics seems remarkable in the light of the fact that the full implications of the effect of water temperature on the fall velocity of sediment particles should clearly point toward the conclusion that everything pertaining to riverbed variation and to the quantities and sizes of sediment put into suspension or motion is affected to a major degree by water temperature. As a final concluding note, it would appear that, when the general thesis of this paper is further confirmed by more exhaustive empirical or laboratory investigations, it may be that hydrologists will adopt the practice of noting discharge at a "standard temperature." Perhaps tables of "correction factors" will be prepared to enable those making discharge computations to reduce the results to a standard temperature basis.

METHODS AND THEIR SUITABILITY FOR DETERMINING TOTAL SEDIMENT QUANTITIES

[Paper No. 32]

By JOHN R. SHEPPARD,[1] *hydraulic engineer, Sedimentation Section, Hydrology Branch, Bureau of Reclamation, Denver*

Abstract

Despite the many advances made in the last 15 years in the investigation and development of hydraulic and sedimentation phenomena, the hydraulic engineer must still use his experience and empirical data to estimate total sediment quantities. In the development of water resources projects, sedimentation is a question that must be answered in one way or another. The answer may be that sediment is of little concern or that a considerable amount of project investigation funds may be required for proper evaluation. In any event, some method must be used with the available data to derive the answer.

The Bureau of Reclamation uses several methods to estimate total sediment quantities that are dependent on the type of data available as well as the suitability of the method to the level of project development. Every effort is made to collect adequate field data for the proper level and to use that corresponding method in deriving the best estimate if possible.

The methods of determining total sediment quantities and a discussion of the suitability of each include: (1) The use of yield rates, (2) the use of suspended sediment sampling data with estimated bedload, (3) suspended data with bedload equations, (4) suspended data with the Modified Einstein procedure, and (5) suspended data with Colby curves of unmeasured load and velocity. A group of Colby curves for a number of western streams are presented.

Introduction

The quantity of total sediment load carried by a stream is a necessary determination in many phases of water resource development. Even if sediment appears to be no problem in planning for a project, it is wise to investigate

[1] Died June 14, 1964.

and estimate the quantities involved. The lack of sediment load, as well as too much sediment load, can create problems. Clear water releases below storage dams, or increased flows in conveyance or drainage channels may cause degradation or erosion of the channel from the lack of a sufficient sediment load.

The sediment load carried in a watercourse has been defined by various component parts. Measured and unmeasured sediment loads are defined and divided by the inability of a suspended sampler to sample near the streambed. Wash load, suspended load, saltation load, and bedload are defined by the predominant mode of transport of sediment particles. The total sediment load is generally understood as the sum of the component loads and is the whole quantity of sediment carried by any and all mode of transport by a stream.

Sediment may be carried by water in any type of conveyance such as pipelines, canals, penstocks, etc. The basic determination of sediment load in project development is that total load carried by a natural stream. For the purpose of this paper, the total sediment load quantities will be those natural loads of an alluvial-type stream that either supplies the water resource for the project or is involved in the project collection, conveyance, or distribution of water.

To the practicing engineer involved in the hydrology of a project, the determination of sediment load quantities is the first and basic step in analysis of sedimentation effects and problems associated with the project features and project operation. Estimates of reservoir storage depletion by sediment, stability studies of conveyance or drainage channels, design of sediment excluding devices at diversion structures, channel degradation estimates below storage structures, studies of channelization design, and other feature studies require the determination or estimate of the total sediment load and its characteristics. The practicing engineer may find it necessary to estimate sediment load quantities for past, present, or future conditions and to evaluate the estimate on the basis of suitability for the level of project development being investigated. The importance of sediment effects on the total project also must be considered in determining the suitability of the method used to derive the total sediment estimate.

A number of methods are described in this paper that have been used by the Bureau of Reclamation to estimate total sediment load quantities for project planning and project operation investigations. Each method has a suitability for certain levels of investigation, which is discussed. To some extent the method selection is also dependent on the type of data available and the expense of a data collection program.

Sediment Yield Rates

Sediment yield rates express the average sediment production of a drainage basin area in terms of a volume or weight per unit area for a unit time period. The usual units are acre-feet per square mile per year or tons per acre per year. Values of yield rates are based on sediment volumes from reservoir or stock pond resurveys, gross erosion rates with sediment delivery ratios, or sampling results on streams. The yield rates derived for storage loss or erosion rates generally are comparable to total sediment load quantities.

Yield rates are used primarily for small drainage areas where the vegetative cover, soils, and rainfall characteristics are uniform. Yield rates are, therefore, used in areas where the sediment produced is a function of drainage basin character rather than stream hydraulics. As such, yield rates can be estimated for drainage basins of similar characteristics.

The use of yield rates is limited to small structures in most cases because of drainage basin size or to appurtenant structures of larger projects. Yield rates are used in cases of tributary cross-drainage sediment contribution to canals where the intermittent flow of the tributary does not warrant a siphon or other cross drainage structure. Yield rates are often used in the preliminary or reconnaissance level of project development, because no other data are available and a data collection program has not been initiated.

The successful use of yield rates is more dependent on the experience of the engineer than upon his technical proficiency in hydraulics. The experience gained by the observation of varied drainage areas with different types of soils and geology, different types of erosion, and different types of vegetation, when combined with rainfall patterns and derived yield rates, is invaluable.

There are many sources of basic yield rate figures, such as the publications of the Soil Conservation Service, Department of Agriculture, on rates of sediment production for various sections of the United States. Any type of stock pond or reservoir resurvey data can provide rates of sediment yield, such as the file of reservoir sedimentation data summary sheets maintained by the Subcommittee on Sedimentation, Inter-Agency Committee on Water Resources. This file contains data from all Federal agencies who are members of the Subcommittee. State and local governmental agencies

may also publish reports that contain data for establishing yield rates.

Suspended Sediment Sampling Data

The largest amount of basic data for estimating total sediment load is the records of suspended sediment sampling measurements on many of our streams. These records report the observed concentration of suspended sediment particles in terms of parts per million by weight or in terms of suspended load in tons per day in conjunction with the observed or measured discharge of water.

With the sampling equipment presently used, suspended sediment sampling measurements do not measure the total sediment load or even the total suspended load. The construction of all samplers in general use at the present time prevents the nozzle from approaching the streambed by a definite distance when the sampler is resting on the bed of the stream. Because the concentration of suspended particles of sand size or larger increases with depth, a portion of the suspended load is not sampled; and the amount unsampled is proportionate to the unsampled depth and the total depth of the stream. The measured suspended load and the unmeasured load are the usual terms that define the total load by the combination of transported load in the two zones of flow.

To derive the average suspended load from the available suspended sampling data, the Bureau of Reclamation generally uses the flow-duration, sediment-rating curve method (*14*). The basic sediment-rating curve is a correlation of water discharge and suspended concentration or sediment load in tons per day. It was first developed by Campbell and Bauder (*1*). A flow-duration curve is a correlation between water discharge and the percentage of time the discharge occurs. Integration of the sediment-rating curve and flow-duration curve will give an average suspended sediment load.

The basic data for the sediment-rating curve and the flow-duration curve do not have to be from the same time period. Because discharge records are usually available over a longer time period than suspended sediment records, this method allows the expansion of a relatively small amount of sediment data to the longer period of discharge, provided no event or control has been imposed on the stream to change the basic sediment-water discharge relation.

The above description is a simple presentation of the method that represents a complex relation. Considerable deviation from a simple curve expressing the discharge–sediment relation is the rule rather than the exception, and may be due to a number of causes. A sediment-rating curve does not necessarily represent the transport capability of a stream but more often represents the water-sediment production capability of the drainage area. Sufficient data to define the average runoff conditions will produce an average curve that can be used with confidence. The water-sediment relation may change with the type of runoff, i.e., snowmelt or rainfall, and should be subdivided into more than one curve for different "seasons" of the time period. Operational changes on controlled streams may also produce different water-sediment relationships requiring subdivision of the sediment-rating curve and flow-duration time periods (*12*). Conservation measures on the watershed or control measures on the stream can produce a pronounced effect on the flow-duration and water-sediment relation.

The suitability of suspended load determinations is primarily dependent on the quality and length of the sediment and discharge data records available. Although a small amount of sediment data can successfully be used to estimate long-period conditions, those data can be biased by the conditions prevailing during the sediment sampling period. Sediment data collected during a low runoff period (compared to the long-period average) may give average loads that are high. Conversely, data collected during a high runoff period may result in average sediment loads that are too low. The suitability is also dependent on the discharge range through which the sampling was conducted in relation to the discharge range of the long-period flow-duration. In determining the average long-period sediment load, the major part is produced by the less-frequent, high discharges. The extension of the sediment-rating curve through this discharge range, because the sampling measurements did not include discharges in the range, can give results that are unreliable.

In project planning, the suspended sediment loads determined with data that may be biased can be estimated or corrected for the beginning levels of project investigation. As the project plan develops to higher levels, additional sampling programs may be desirable to correct the bias or obtain data through the greatest possible range of discharges.

The major part of available suspended sampling data is found in U.S. Geological Survey Water Supply Papers, although other Federal, State, or local agencies may conduct sampling programs to fill their needs.

Reports of published and unpublished sediment data inventories are issued periodically by the Subcommittee on Sedimentation, Inter-Agency Committee of Water Resources.

With the determination of suspended load estimates, unmeasured load estimates are

necessary to determine total sediment load quantities. Various methods are presently used that include estimating bedload, bedload equations, total load methods, and extension of total load method results.

Estimating Bedload or Unmeasured Load

E. W. Lane and W. M. Borland presented a paper dealing with estimating bedload, which is a suitable guide (*8*). The following discussion and table are quoted from that paper.

The usual method of determining the amount of bedload correction is to assume that it is a certain percent of the load determined from the suspended load sediment samples. The percentages which have been used in the past have been based on judgment, which in many cases have been little better than a guess. Opinions differ considerably on the correct value of this percentage to assume. For different streams, percentages which have been used by various engineers range from 0 to 100 percent, and it is probable that the correct value for these streams covers a range of at least this magnitude. Very few quantitative measurements of the total load have been made to serve as a guide in making accurate estimates of bedload, but there are certain general relations which, if kept in mind, will greatly narrow the limit of error which an engineer is likely to make in estimating the bedload. There are three major variables which affect the amount of bedload a stream may carry. They are as follows:

1. Size of bed material or the fall velocity of these particles.

2. Slope of the stream or the average stream velocity.

3. Nature of the channel, including the depth, size, shape, and roughness of its bed and banks.

A minor factor is temperature, including the effect of ice. To aid in the evaluation of the effect which these variables have on the bedload the following criteria are set forth:

Criterion 1. The smaller the actual concentration of suspended material, the higher usually will be the percent of bedload. This is a very important factor in estimating bedload.

Criterion 2. The smaller the difference in the particle sizes of the bedload material and the suspended load material, the higher will be the percentage of bedload to the total load.

Criterion 3. The ratio of bedload to suspended load is apt to be larger for low or medium stages than it is for high stages. Thus, a stream in which the flow does not fluctuate widely, is likely to carry a larger percent of bedload. This criterion does not necessarily apply to very steep mountainous streams which move boulders at flood stage. In such cases, the bedload ratio may increase with an increase in discharge.

Criterion 4. Streams with wide shallow channels carry a higher proportion of sediment as bedload than streams with deep narrow channels.

Criterion 5. Stream channels with a high degree of turbulence tend to have smaller amounts of bedload.

A clue to the magnitude of the bedload correction can often be found in the nature of the source of the sediments involved. The three common types of erosion which produce sediment are: 1. Sheet erosion, 2. gully erosion, and 3. bank cutting. Sediment produced by sheet erosion tends to be fine; and, therefore, streams carrying sediment derived from this source tend to have a small amount of bedload. However, if the topsoil is predominantly sandy and the drainage-ways are comparatively small, then shallow-sheet erosion may produce considerable bedload. Gully erosion and bank cutting are likely to produce coarser material and, therefore, more bedload than is obtained from sheet erosion. However, if the material below the surface layer of the ground is fine or is composed of material which breaks down into fine material, as does shale, gully erosion or bank cutting do not produce a large amount of bedload. Since bedload is composed of the coarser particles, in order to have an appreciable amount of bedload, these coarser particles must be found in abundance on the watershed and must be so located that they can be easily moved in to the drainage-ways of the stream's channel system.

For some purposes it is desirable to estimate the bedload movement at a given time, and for others it is necessary to estimate the average movement over a period of years. Most of the conditions previously discussed deal more particularly with the conditions at a given time. When the average bedload over a period of time is concerned, the flow variability must be considered. Most of the sediment load of streams is carried during high-flow periods since at these times both the discharge and the sediment concentration are high. At times of high flow, however, the depth of flow and turbulence are greater and tend to lower the bedload content of the stream (at least in the case of the sand sizes, the amount of which is likely to be of greater magnitude than the coarser material). The bedload correction in times of high flow is, therefore, likely to be a smaller part of the suspended load than in times of low flow; consequently, with a more variable flow the bedload correction will be smaller, when expressed as a percent of the total quantity of measured suspended load for the period of flow involved.

From the foregoing discussion it can be seen that a great many factors are involved in the estimation of bedload and that it is difficult, if not practically impossible, to draw up a simple rule or formula which will give quantitative answers to all cases. Probably the best answer to the problem devised to date is that suggested by Thomas Maddock, Jr., and given in Table 1. This table takes into account only a portion of the variables previously mentioned, but those omitted can be given consideration in determining that part of the suggested bedload range to adopt.

Bedload Equations

Numerous bedload equations have been proposed over the years, such as Einstein (*4*), Schoklitsch (*13*), Kalinske (*7*), Meyer-Peter and Muller (*11*), Straub (*17*), and many others. It is necessary for the practicing engineer to have some knowledge of the data, methods, derivation, and limitations of the various bedload formulas before they are applied to a particular situation. Most of the formulas are based on results of the tractive force flume studies of Gilbert, Kramer, the U.S. Waterways Experiment Station, and others. The Schoklitsch (*13*) formula, however, was developed for coarser materials on European streams and was verified by measurements on these streams. The main deterrent to verification of most of the formulas in natural stream channels is the lack of suitable methods for measuring the actual bedload movement. The Geological Survey has made several studies where natural or artificial contracted sections have been used and the created turbulence was sufficient to

TABLE 1.—*Maddock's classification for determining bedload*

Concentration of suspended load	Type of material formingthe channel of the stream	Texture of the suspended material	Bedload in terms of suspended load
Parts per million			
Less than 1,000	Sand	Similar to bed material.	25 to 150
Less than 1,000	Gravel, rock, or consolidated clay.	Small amount of sand.	5 to 12
1,000 to 7,500	Sand	Similar to bed material.	10 to 35
1,000 to 7,500	Gravel, rock, or consolidated clay.	25 percent sand or less.	5 to 12
Over 7,500	Sand	Similar to bed material.	5 to 15
Over 7,500	Gravel, rock, or consolidated clay.	25 percent sand or less.	2 to 8

force most of the normal bedload into suspension where it could be sampled (*3,6*). Such contractions are not available on a sufficient number of streams of varied character to establish comprehensive criteria for bedload formula use, however.

Several formulas that apply to a situation can be used and the separate results compared. In this way the limitations and variance of each is developed by experience. As an example, the Schoklitsch formula, using a mean depth of section, and the Meyer-Peter and Muller formula, using the maximum depth of section, generally result in comparable bedload values for a given set of conditions. This is likely the result of data used in formula derivation; Schoklitsch based on natural European streams and Meyer-Peter and Muller based on laboratory flume studies.

An additional problem is created by the use of bedload formulas to represent the unmeasured load of a natural stream. Most basic data on which the formulas are based are the quantities of transported material. The flume experiments when conducted through a series of discharges did not consider that higher discharges would carry an increasing part of the bed material as suspended load. A formula based on such data could compute more than the unmeasured load because, if all other conditions were the same, the bed material in suspension would be computed as unmeasured load and also sampled as measured load.

Even though there are limitations to the use of bedload formulas, they are adaptable to changing hydraulic conditions in the stream channel and can be used to estimate loads for a variety of problems associated with project development and planning.

Comprehensive Total Load Methods

There are several methods, used in present practice, that compute the total load of a stream as an integrated load of both suspended and bed transport. E. M. Laursen developed parameters by descriptive analysis that are involved in sediment transport and established relationships between the parameters from approximations and empirical definitions (*9*). However, use of the Laursen computational procedure has not been successful on some major natural streams without revision (*10*).

The Modified Einstein procedure is another computational method which results in an estimated total sediment load (*3, 15*). The unique feature of this method, and possibly one of the main reasons for its acceptance among field engineers, is the use of field sampling measurement data in the computation. The Laursen computation used hydraulic and sediment parameters to result in a total sediment concentration. The Modified Einstein procedure uses the sampled concentration of suspended sediment and expands or integrates to determine the rest of the total load. The total load results by the Modified Einstein procedure are correlated to observed data, but the Laursen method computes what can be directly measured.

The Modified Einstein method can be used only in the middle range of sediment transport; i.e., where the transported materials are primarily the same in suspension and along the bed. To integrate the suspended load through the full stream transport depth, including bedload movement, the same size of particles must be found in both modes of transport. If the suspended particles are appreciably smaller than the bed material, the particles are probably transported as wash load. In such a case, separate computations of bedload and suspended load are warranted instead of a comprehensive total load method.

The necessary data and measurements required for a Modified Einstein computation are greater than those for a suspended sediment sampling program, because sampling of the streambed material and a size analysis of both bed and suspended particles are necessary. Perhaps the largest increase in cost of a total load program in comparison to a suspended sampling program is in the time and expense necessary to make the Modified Einstein computations. An engineer or engineering technician will require 2 to 4 hours to make one computation by hand. The Bureau of Reclamation has developed the Modified Einstein procedure for use on an IBM 650 electronic computer, thus saving engineering time but still requiring additional expense (*16*). From these

considerations, it should be clear that a total sediment load program can be economically justified only for problems of considerable importance or for project planning on the higher design levels.

The end result of a Modified Einstein computation is a total sediment load for one discharge value similar to the suspended sediment load for one discharge value of a suspended sediment sampling. The methods used to derive average loads as described for suspended sediment sampling data are, therefore, equally adaptable to total load data. The same limitations and qualifications are also applicable.

The critical determination that affects the total sediment load in a Modified Einstein computation is the rate of change of particle concentration with depth denoted as the Z value. Einstein (*5*) has expressed the concentration at any distance y from the bed, C_y, as a function of a known concentration at distance a as follows:

$$C_y = C_a \left(\frac{d-y}{y}\,\frac{a}{d-a}\right)^Z$$

Various methods have been developed to determine the Z values for various size fractions of the total load. Einstein (*5*) proposed the use of the formula:

$$Z = \frac{V_s}{0.4u_*}$$

Point sampling measurements have shown the Z value from this form of equation is too high. Corresponding Z values have been determined at natural sections for measurements at contracted sections by the U.S. Geological Survey. The U.S. Bureau of Reclamation has attempted point sampling measurements on the Colorado River, which have not been successful. The Modified Einstein procedure determines Z values by computing a concentration C_a near the bed level by the bedload function and integrating across the unmeasured depth to agree with the measured suspended concentration. None of these methods have proved completely successful. Until some method is devised to predict the Z value for other size fractions from the Z value of one size fraction and this method agrees with sampling data, the Modified Einstein computation will continue to contain inconsistencies.

The Corps of Engineers sampling program on the Missouri River has been devised to compute total suspended load by the collection of an equal transit rate sample for the whole stream and a point sample in each vertical. These two samples define a measured Z value for extending the suspended concentration to the bed, but the method does not include any bedload transport. This method has been adopted because, in the Missouri River, bedload is not a large part of the total transport. Some major streams of concern to the Bureau of Reclamation, such as the lower Colorado River, carry an average of 40 percent unmeasured load, and the approximation by the Modified Einstein procedure is considered a better estimate for these streams than a total suspended load value.

In any case, much more investigation is needed to establish the factors affecting the rate of concentration change with depth in natural streams. The Modified Einstein method is still useful as it is, because the Z values for the size fractions that make up the major part of the total transport can be determined to some degree of accuracy, although the relation between the various Z values is somewhat vague.

Velocity and Unmeasured Load Curves

B. R. Colby presented a relation of unmeasured sediment load to mean stream velocity as an approximation for determining total sediment load (*2*). The Bureau of Reclamation has checked this relation on a number of streams where Modified Einstein computations were available to derive such a curve. The main premise of the Colby relation is that most of the variables used in the Modified Einstein procedure have a relation or are affected by the mean stream velocity. The unmeasured load should, therefore, show a strong correlation with mean velocity. The stream width has been eliminated in this correlation by using the unmeasured load per foot of width.

Available total load results by the Modified Einstein procedure were used to determine individual curves of mean velocity vs. unmeasured load per foot of width for the following:

Rio Grande:
- Albuquerque, N. Mex.
- Belen, N. Mex.
- San Felipe, N. Mex.
- Bernalillo, N. Mex.
- Cochiti, N. Mex.
- Miscellaneous measurements.

Colorado River:
- Northerly International Boundary.
- Adobe Ruin.
- Taylor's Ferry.
- Palo Verde Dam.
- Water Wheel.
- R. S. 43.
- R. S. 41.
- Needles Bridge.
- R. S. 33.
- Columbus, Tex.
- Bay City, Tex.

Guadalupe River:
Victoria, Tex.
Sabine River:
Bon Weir, Tex.
Trinity River:
Romayor, Tex.
Brazos River:
Richmond, Tex.
Elk Creek:
Hobart, Okla.

Figure 1 shows a composite plot of the derived curves for the listed gaging stations, although the individual curves cannot be discerned.

The 57 Modified Einstein computation results for the Rio Grande at Cochiti, N. Mex. (fig. 2) were processed for a statistical correlation to approximate a curve of the following form:

$$Y = CX^D$$

From the correlation, it was found that 90 percent of the deviation of unmeasured sediment load from the curve could be explained by velocity variation. From statistical variation, 18 of the points would be expected to fall outside one standard deviation from the curve. For the Cochiti correlation, only 14 of the 57 values fell outside one standard deviation.

In recent studies of sediment movement in the Lower Colorado River at the northerly International Boundary, the flow-duration, total sediment load curve method of computing the yearly loads, was checked by using suspended sediment data and a derived curve of mean velocity vs. unmeasured load per foot of width based on 50 total load computations (fig. 3). Over a period of 5 years, the total accumulated sediment load determined by the two methods differed by only 3.5 percent.

Based on the previous studies and additional experience with unmeasured load curves, the Colby relation appears to be a valuable tool for use in total load estimates. The basis of the relation is total loads as determined from Modified Einstein computations, and any application should be in a situation where the Modified Einstein method is applicable. It does allow a total load investigation with less data than necessary to establish a total sediment load rating curve and can approximate conditions for similar streams where no data are available. However, sufficient data should be collected to define such curves based on the level of project development being investigated.

Additional unmeasured sediment load curves are shown in figures 4, 5, 6, 7, 8, and 9 for several of the listed gaging stations. It is hoped

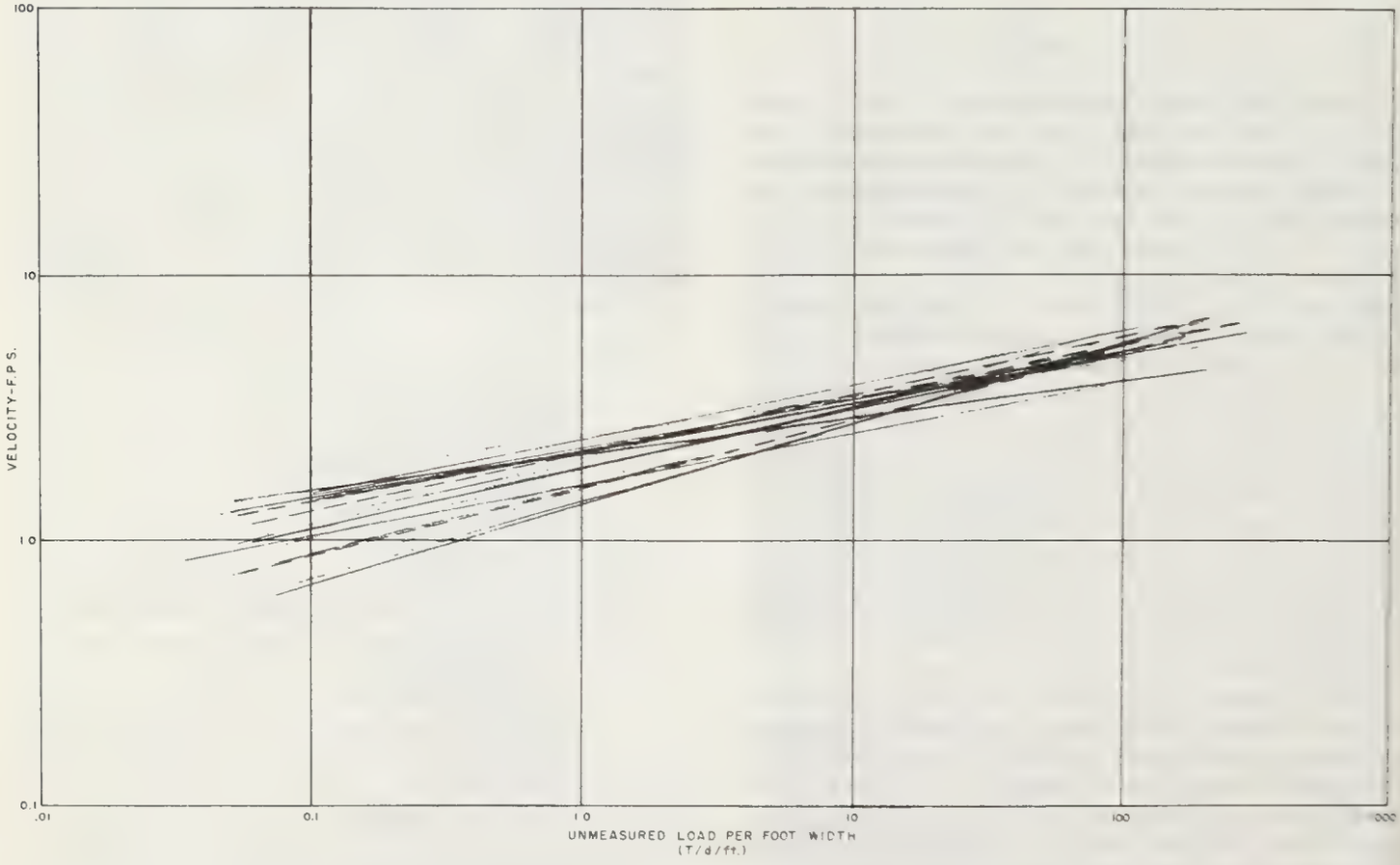

FIGURE 1. — Composite plot of relation between unmeasured sediment load and mean velocity.

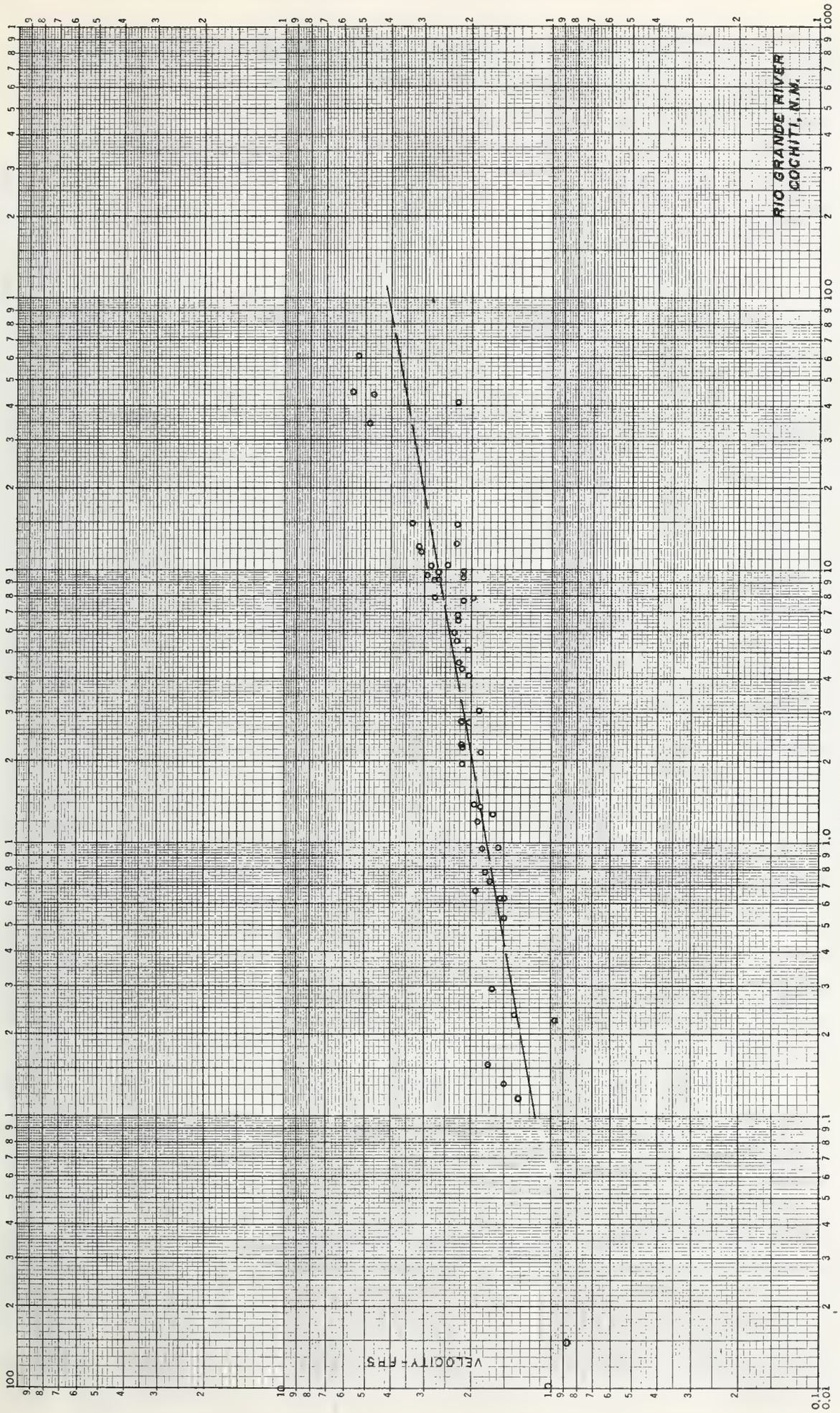

FIGURE 2. — Relation between unmeasured sediment load and mean velocity, Rio Grande, Cochiti, N. Mex.

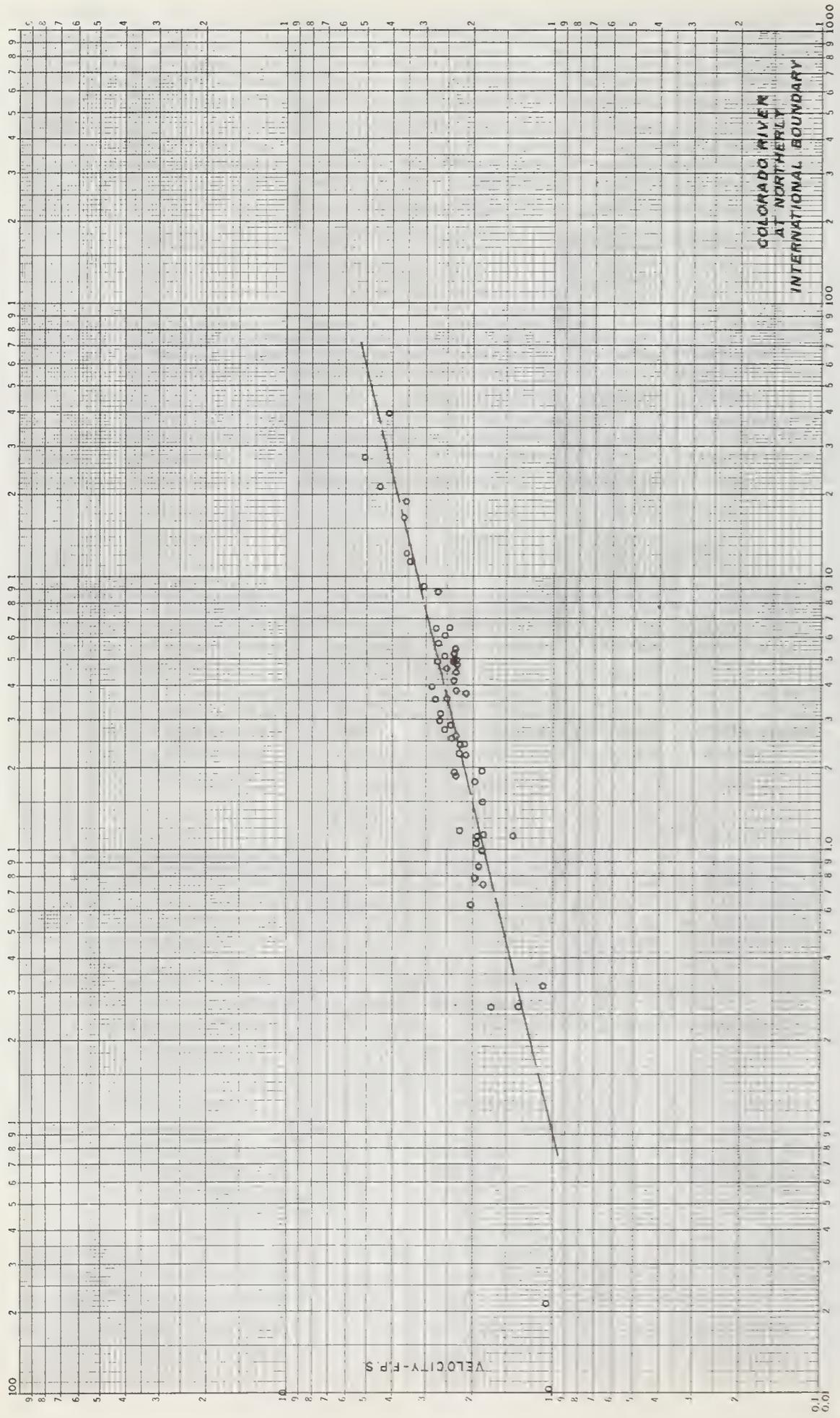

FIGURE 3. — Relation between unmeasured sediment load and mean velocity, Colorado River at northern International Boundary.

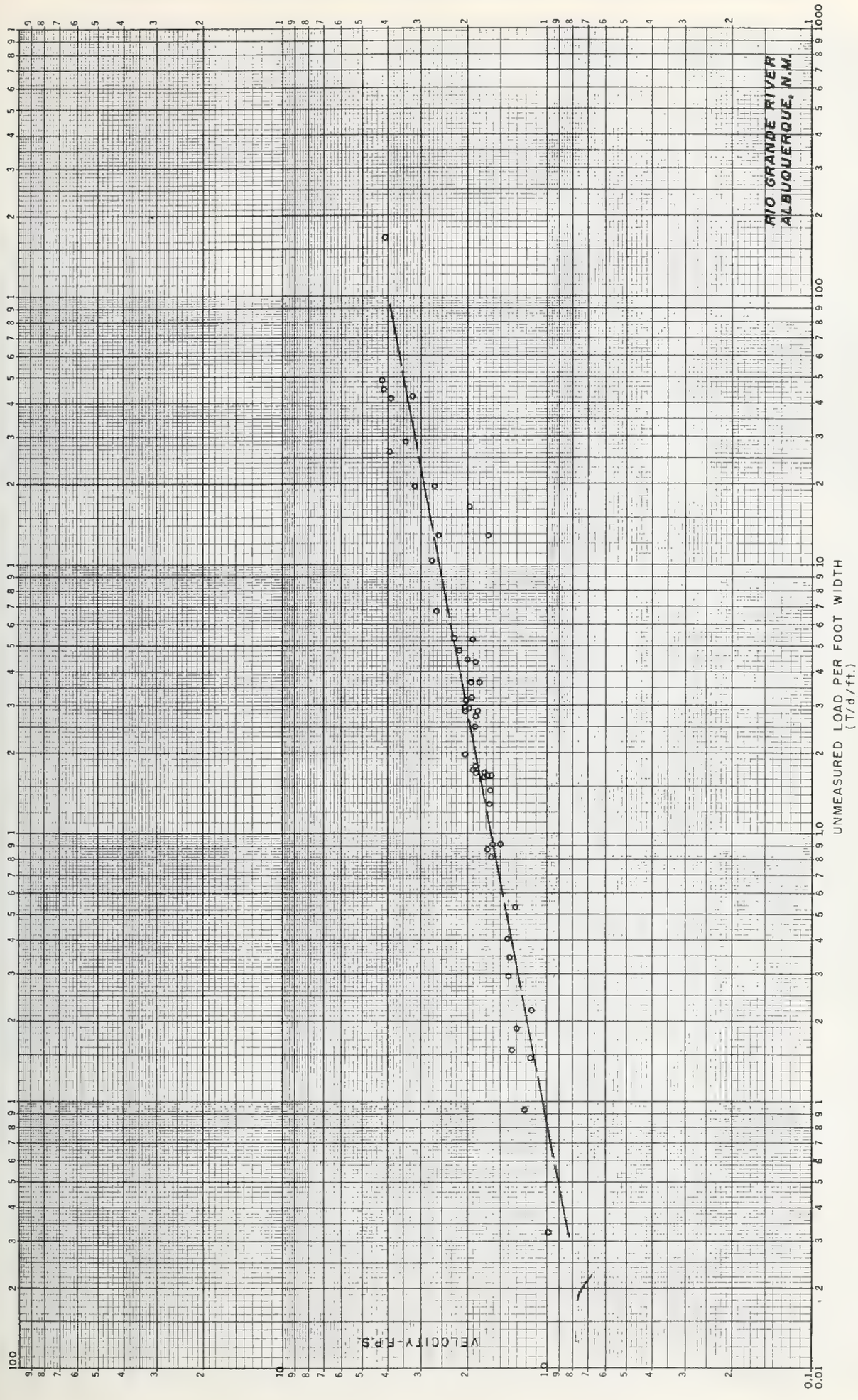

FIGURE 4. — Relation between unmeasured sediment load and mean velocity, Rio Grande, Albuquerque, N. Mex.

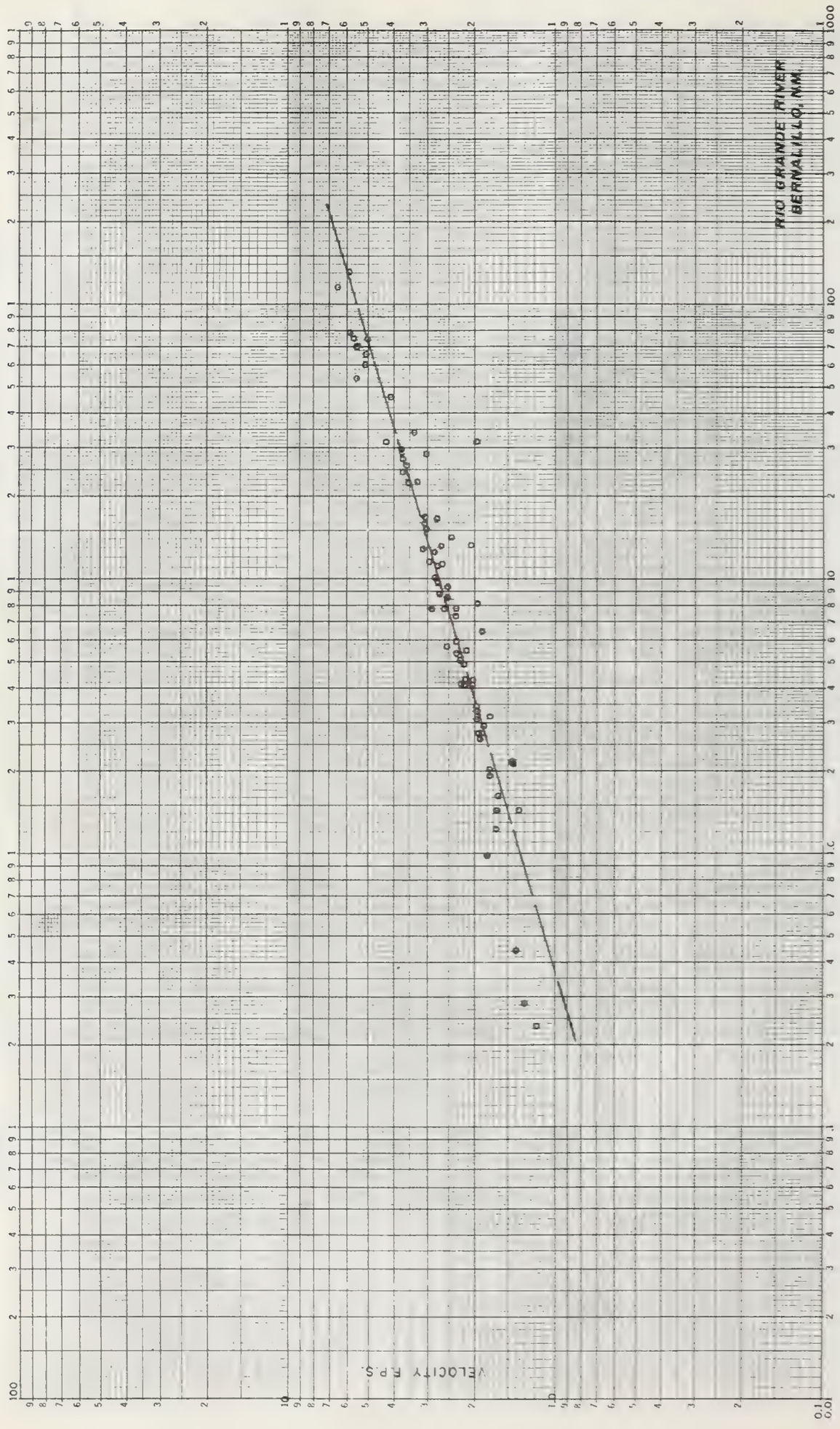

FIGURE 5. — Relation between unmeasured sediment load and mean velocity, Rio Grande, Bernalillo, N. Mex.

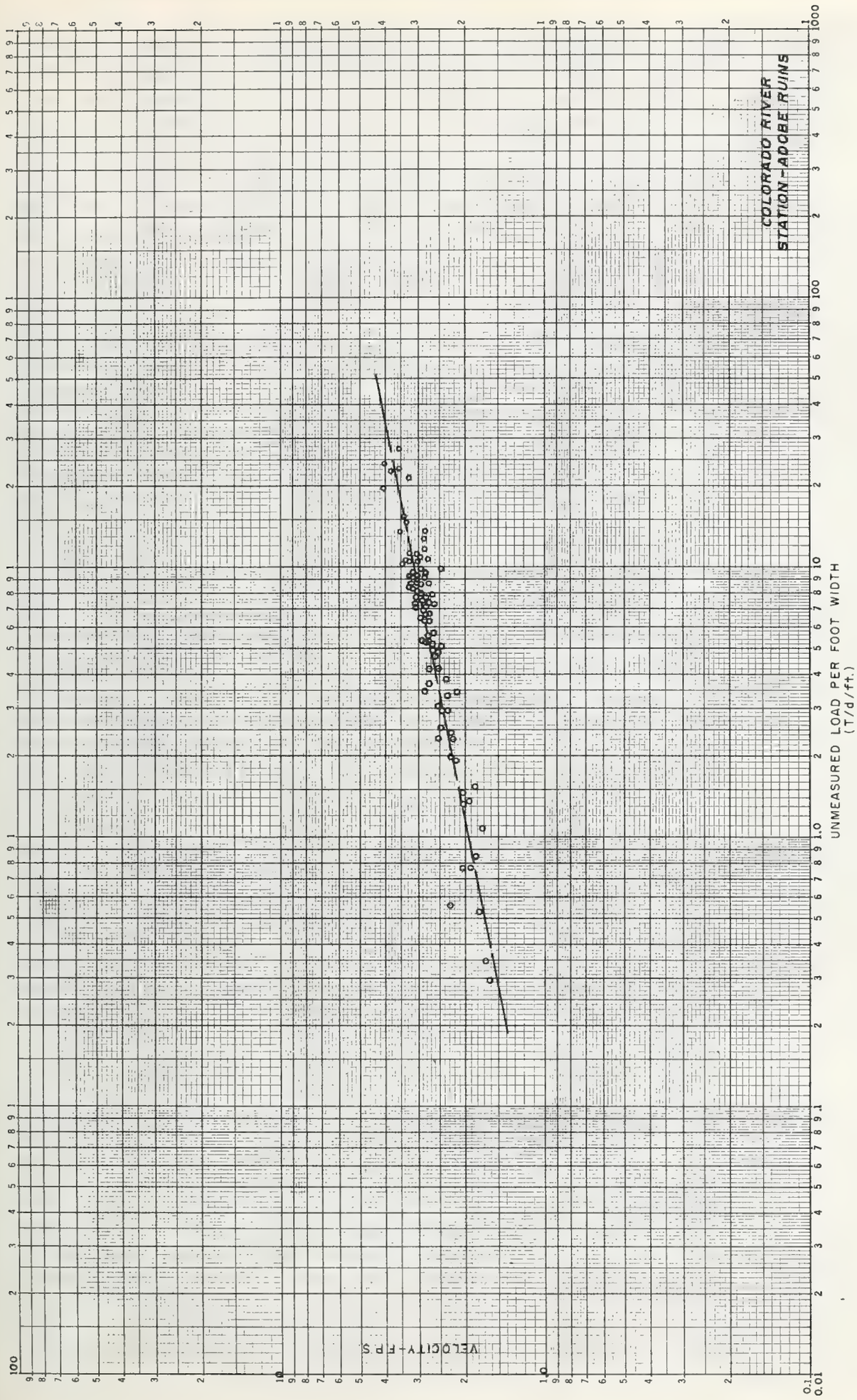

FIGURE 6.—Relation between unmeasured sediment load and mean velocity, Colorado River, Adobe Ruin Station.

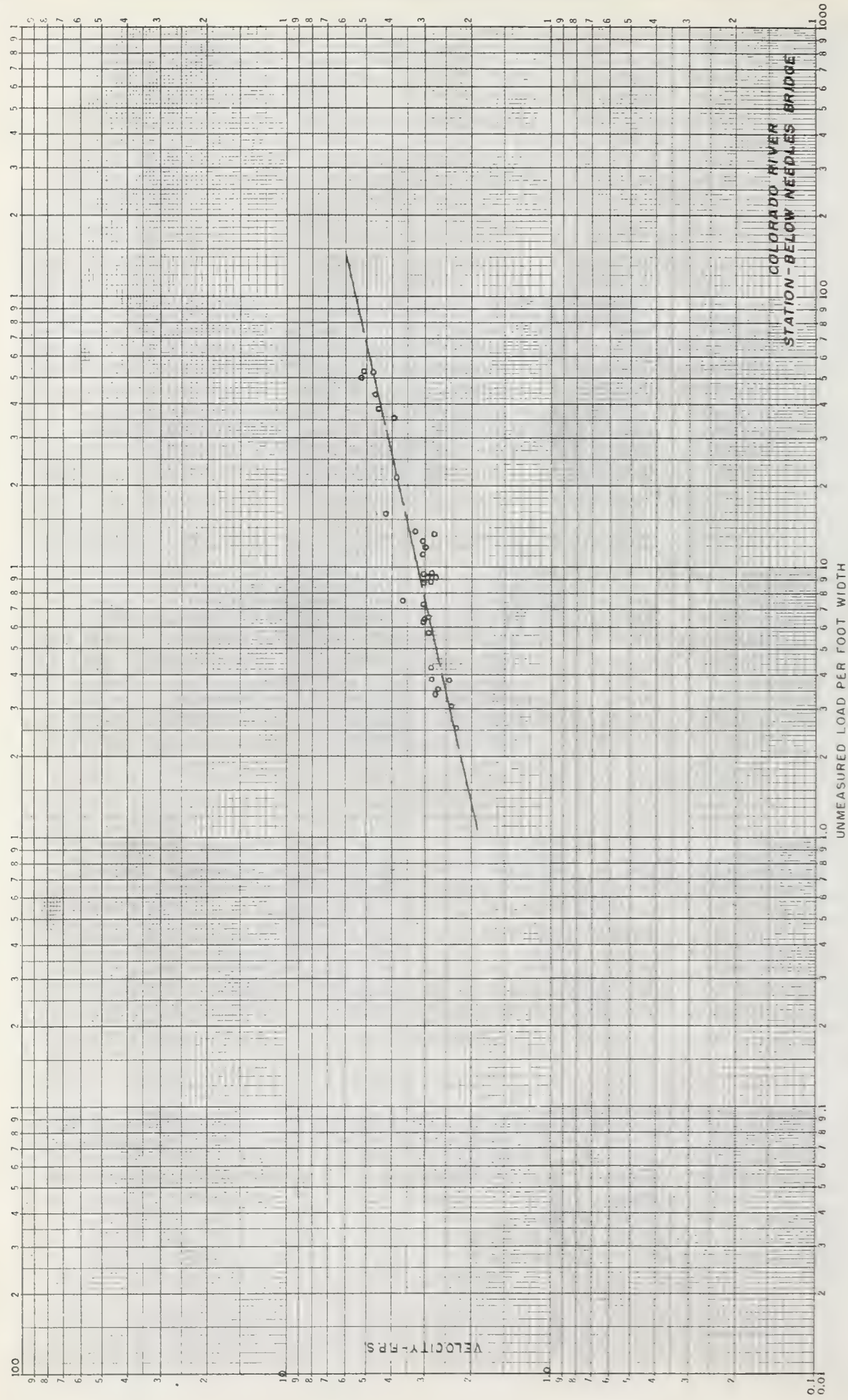

FIGURE 7. — Relation between unmeasured sediment load and mean velocity, Colorado River, station below Needles Bridge.

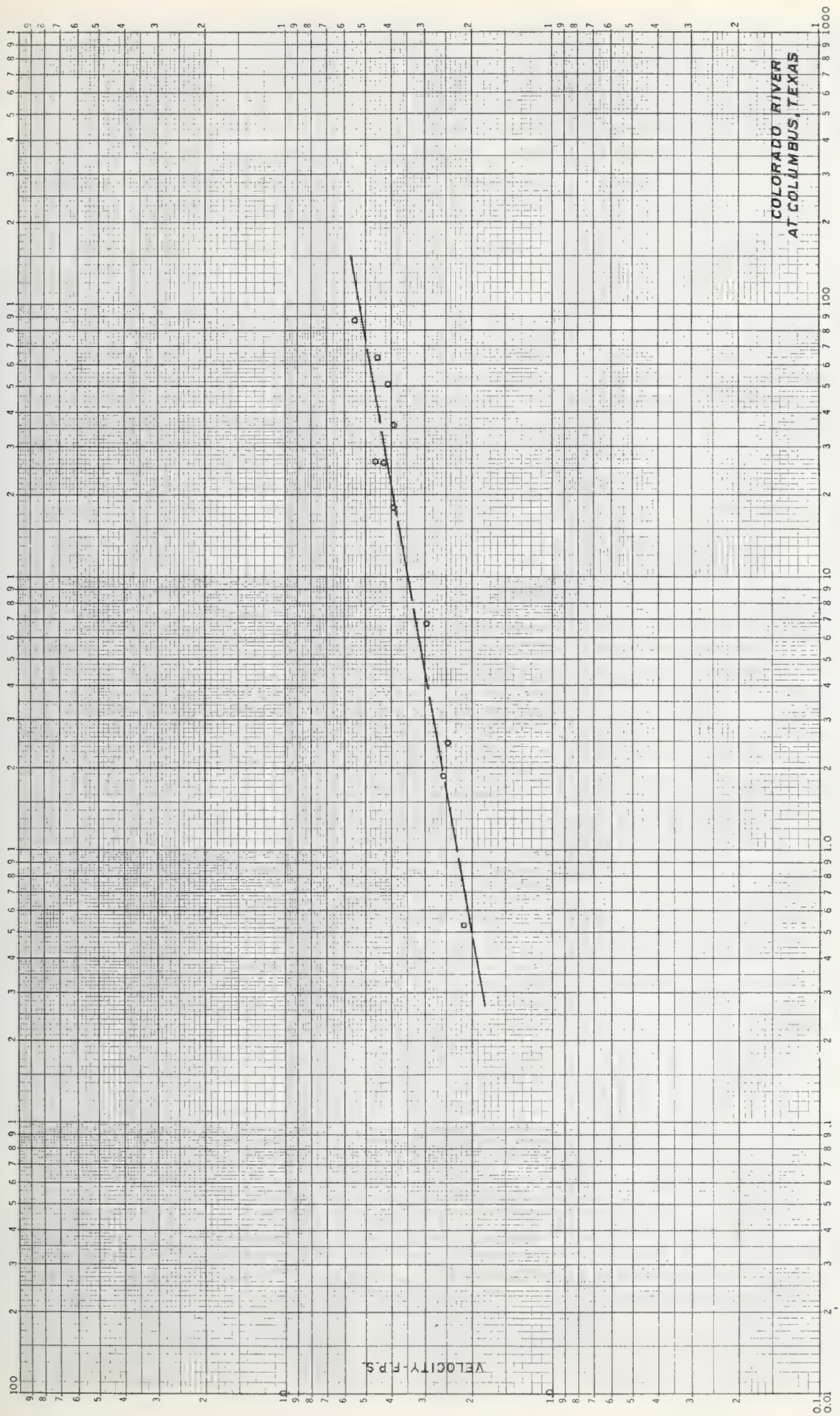

FIGURE 8.—Relation between unmeasured sediment load and mean velocity, Colorado River, at Columbus, Tex.

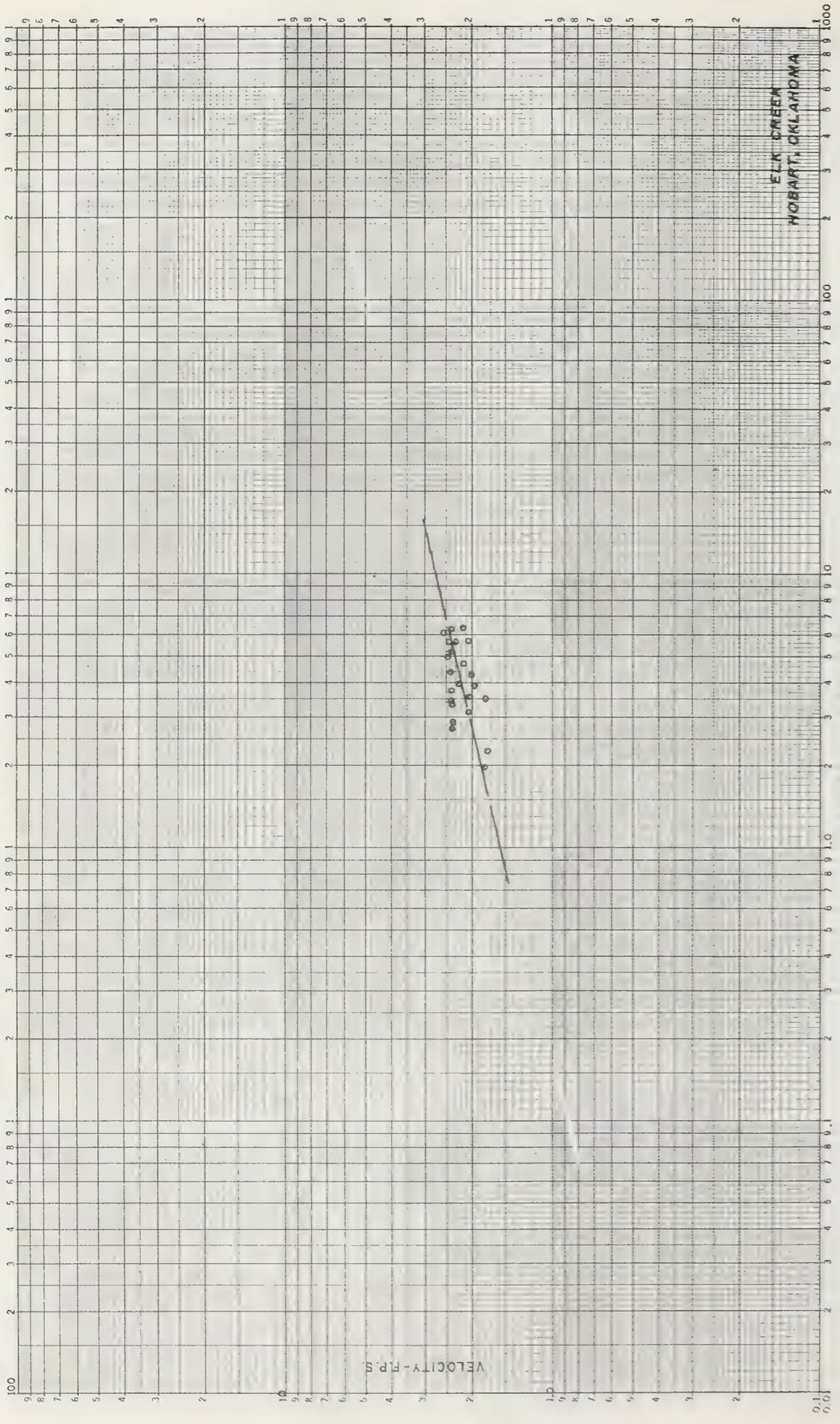

FIGURE 9. — Relation between unmeasured sediment load and mean velocity, Elk Creek, Hobart, Okla.

that in the future, the series of curves shown in figure 1 may be studied to determine variation in slope and position with additional stream or sediment characteristics.

Conclusion

The determination of total sediment quantities during the investigation phase of a water resource project is an important and often vital part of the design and operation of the project. Sediment can produce problems with major or minor structures where the transport of water is involved. This paper has been prepared as a guide to the methods generally used in Bureau of Reclamation practice to determine total sediment quantities in natural streams. The main purpose has been to present these methods not in complete detail, but in the frame of ideas for a practical determination of quantities. Sufficient references have been included if the reader wishes to study the methods in detail.

The prediction of greatly increased water resource needs for the future carries with it the implication of continued and increased concern with transported sediment. The methods presented in this paper may help in this concern, although new investigations, advancements, and studies will be necessary to provide the practicing sedimentation engineer with the necessary tools.

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SEDIMENT CONTROL AT A HEADWORKS USING GUIDE VANES

[Paper No. 33]

By E. J. Carlson, *head, Sediment Investigations Unit,* and P. F. Enger, *hydraulic engineer, Hydraulics Branch, Division of Research, Bureau of Reclamation, Denver*

Synopsis

The Bureau of Reclamation has in progress a general study program concerning the control at canal headworks of coarse sediments diverted from alluvial streams. The study described in this paper was conducted as one phase of the general program. Its purpose was to develop satisfactory arrangements of bottom and surface guide vanes, to compare the performance of each type, and to evaluate the effectiveness of each type in reducing sediment intake into a canal diverted from a large river. Thirty-seven tests were made on one type of canal entrance with a standard river and diversion discharge.

Tests indicated that both bottom and surface

vanes, as shown in figures 7 and 9, are effective in reducing sediment intake into a canal diverted from a large river. With either bottom or surface vane operation, it was possible to reduce the quantity of sediment entering the model canal to approximately one twenty-third of the quantity entering without the vanes in place.

Because the vanes are not overly sensitive to the tested variables, including length, spacing, placement, and depth, the information in this paper should be of value in design work. For intake conditions similar to those used in these tests, guide vanes having dimensions proportional to those given in figures 7 and 9 should prove beneficial. Results of these tests show that guide vanes to control sediment movement can be developed by means of model tests.

Introduction

Sediment removal from canals, laterals, and farm ditches is costly maintenance, so that it is desirable, if possible, to exclude or reduce the amount of sediment going into a canal headworks that takes water from a sediment-laden stream. Guide vanes placed near a headworks or sluiceway entrance have been studied [1] as one effective method of controlling sediment movement near the intake. They are used to control localized secondary currents (figs. 1 and 2) by diverting bottom water with its relatively heavy sediment load away from the canal headworks, and top water with its relatively light sediment load through the canal headworks.

Studies to develop and evaluate the effectiveness of bottom guide vanes for a particular arrangement of intake were made for the San Acacia Diversion dam and are reported in the Bureau of Reclamation, Hydraulics Branch Report Hyd–479.[2] A bottom vane arrangement, determined by model tests at the Denver laboratory that resulted in satisfactory performance is described in this paper. At the conclusion of the San Acacia tests, the model was used to extend the data on surface and bottom guide vanes as a research project. Dimensions referred to in this paper are for the San Acacia dam. Prototype dimensions are used in the narrative because they are easier to visualize. Some prototype dimensions are shown in terms of a model scale of 1: 20, in parentheses on the figures, but most of the dimensions are model dimensions and show the actual size of the test facility.

Although the scope of these studies is limited, the research indicated that efficient guide vanes can be developed by means of model studies and that additional research would provide valuable generalized design information.

Experimental Work and Data Obtained

The model (figs. 3 and 4) was constructed in a test box lined with sheet metal. The prototype spillway consisted of 12 river bays with 20-foot by 7-foot 6-inch gates and an adjacent sluiceway area. A movable bed extended approximately 600 feet upstream from the dam axis. The canal headworks was approximately 160 feet upstream from the dam axis and discharged into a canal of 8-foot bottom width and 2:1 side slopes. The low-flow channel used in the San Acacia study (fig. 3) with the five entrance conduits, was not operated in these tests.

Such major features as river gates, conduits, slide gates, and sampling equipment were generally constructed of sheet metal. Treated wood was used for piers between radial gates, and a part of the canal was constructed of metal lath covered with concrete.

A fine sand of near uniform size gradation (fig. 5,*A*) was used to form the movable bed and to represent sediment bedload in the model. The average diameter of the model sediment was approximately 0.2 mm. (millimeter). Figure 5,*B*, shows the settling velocities of the model sand.

Two pumps were used to supply water and sediment to the model. No. 1 pump, positioned at the downstream end of the model (fig. 3), recirculated sediment-laden water through the model. No. 2 pump drew clear water from a laboratory reservoir and supplied a small quantity of water to replace sampling and other losses. This water was introduced into the upstream end of the model and maintained a constant head on pump No. 1. Excess water discharged over a weir at the downstream end of the model. Discharges and water surface elevations in the canal were maintained constant by the use of slide gates at the downstream end of the canal. Backwater was maintained on the radial river gates by the use of the slide gates installed for this purpose downstream from the diversion dam spillway (figs. 3 and 4).

Samples of water and sediment discharging from the canal, the sluiceway, and the river gates were obtained by passing a hand-operated sediment sampler through the discharging water (fig. 4,*B*). The sediment-laden water flowed through the sampler to a volumetric col-

[1] POTAPOV, M., and PYCHKINE, B. METHODS OF TRANSVERSE CIRCULATION AND ITS APPLICATION TO HYDROTECHNICS. Moscow Academy of Sciences, U.S.S.R. Translation 46 of Service des Etude et Recherches Hydrauliques, Paris. 1947.

[2] BUREAU OF RECLAMATION. HYDRAULIC MODEL STUDY TO DETERMINE A SEDIMENT CONTROL ARRANGEMENT FOR SOCORRO MAIN CANAL HEADWORKS—SAN ACACIA DIVERSION DAM, MIDDLE RIO GRANDE PROJECT, NEW MEXICO. Hydraulics Branch Report Hyd-479. March 1962.

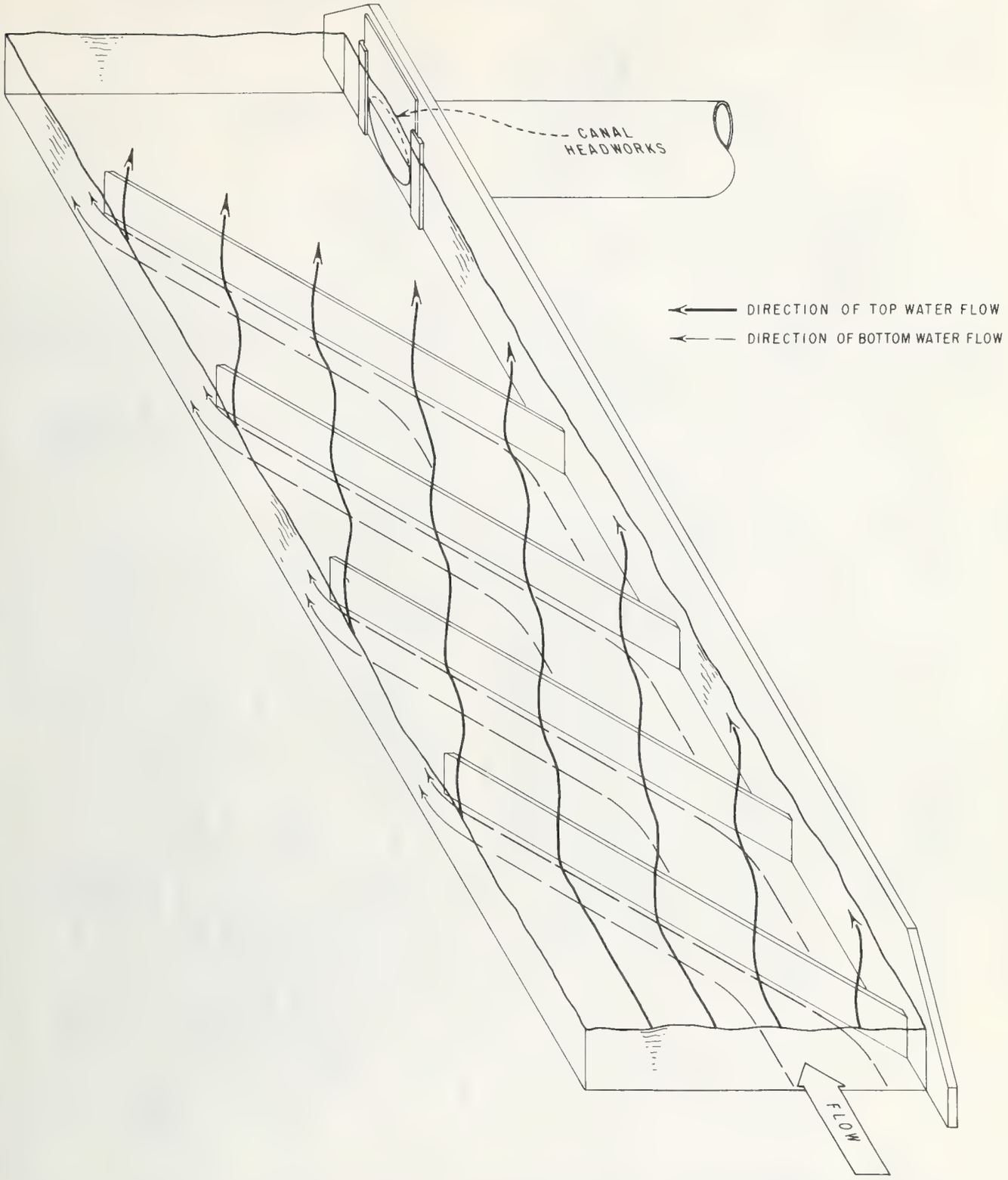


FIGURE 1. — Bottom guide vane method of producing secondary currents for sediment control.

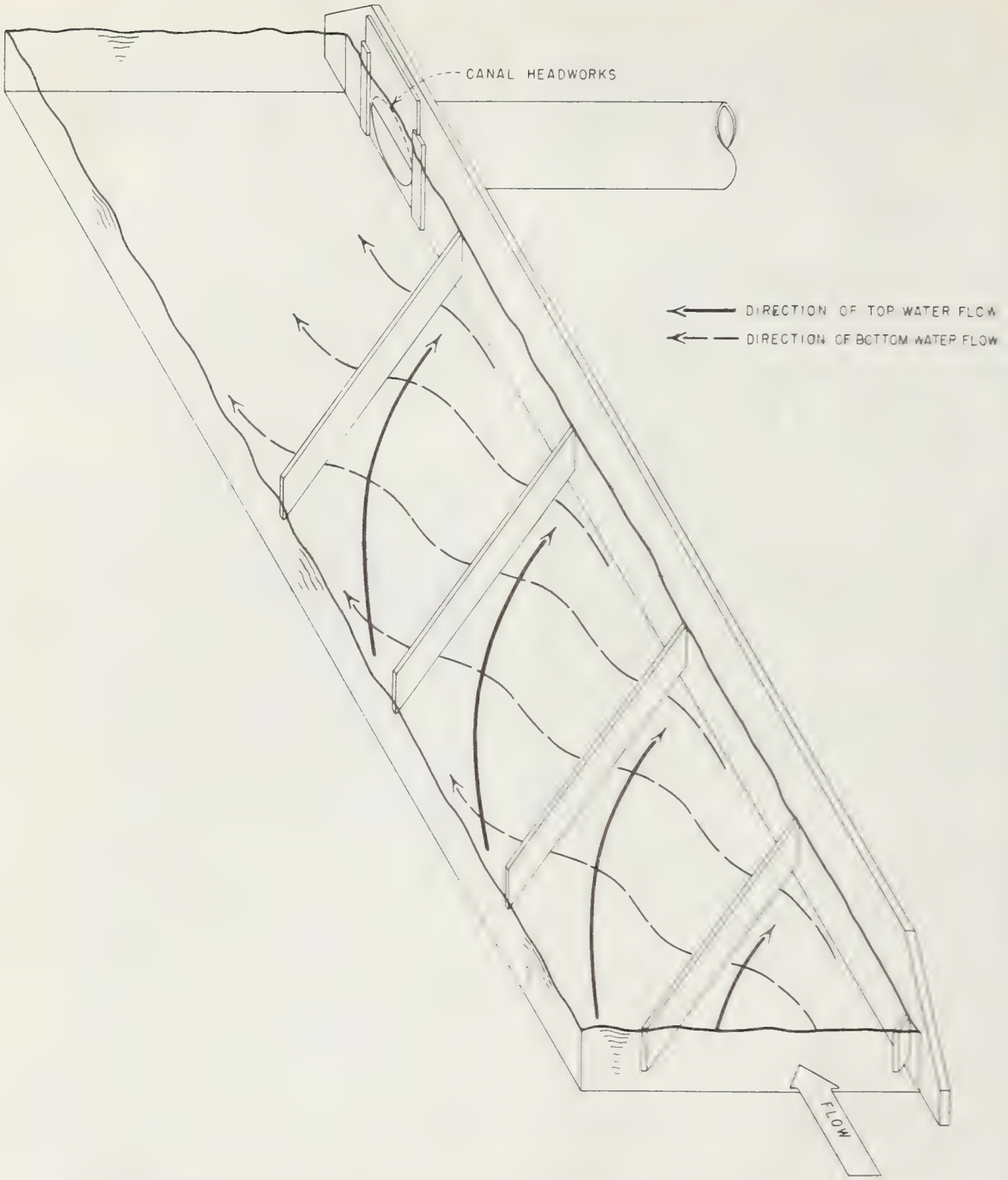


FIGURE 2. — Surface guide vane method of producing secondary currents for sediment control.

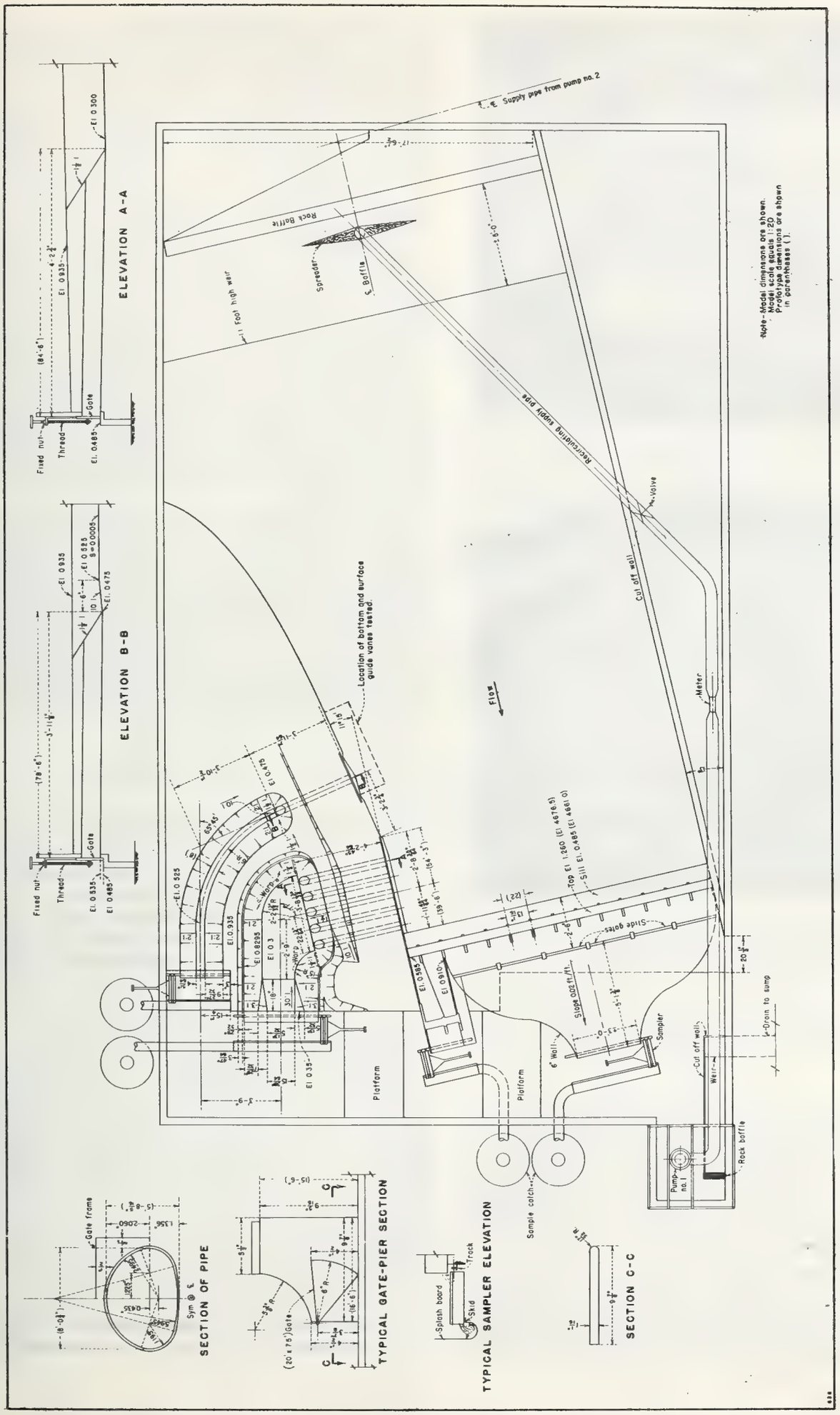
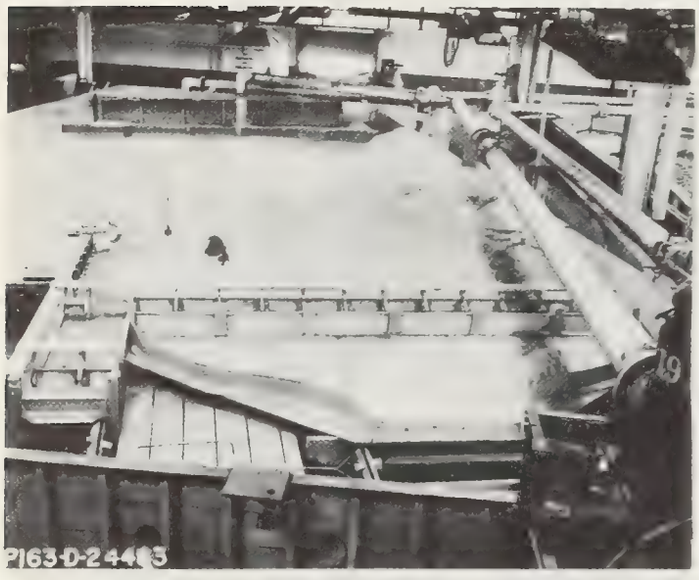


FIGURE 3.— General plan and sections of hydraulic model.



(a)
Overall View of Model

(b)
Sample Being Obtained from River Flow

FIGURE 4. — *A*, Overall view of model; and *B*, sample being obtained from river flow in hydraulic model study.

lector calibrated to indicate the amount of water and sediment in liters. After the sediment had settled into the small funnel at the bottom of the collector, its volume was determined. Thus, the concentrations of sediment passing through the sluiceway or the canal could be readily determined at any time during a test.

During a test some sediment deposited and remained in the canal. To account for these deposits that had entered the canal but had not been accounted for in the sampling process, the volume of sediment remaining in the canal was also measured. The sediment concentration was therefore based on the discharge and the average sediment concentrations that passed through various parts of the model, taking into account the amount of sediment deposited in the canal.

For all tests, the water surface elevation just upstream from the dam was held at 4,668.7 feet; the tailwater elevation below the radial river gates was held at 4,667.6, to correspond to the tailwater used in the San Acacia model study. The canal intake gate was calibrated while holding the headwater at normal elevation 4,668.7 and maintaining the canal water surface at the calculated normal elevation for the discharge. A standard test discharge, similar to that used on the San Acacia model study, consisting of 8,760 cubic feet per second in the river and 174 cubic feet per second diverted to the canal, was used throughout the study (1:20 model scale).

When contours of the movable bed configuration at the end of a test were desired, levels were obtained and appropriate plots made. To help evaluate results, both black and white and color photographs of sediment deposits and bed conditions were obtained for each test. Tests were compared on the basis of a ratio of the concentration of sediment entering the canal headworks to that moving in the river upstream from the headworks:

$$R = \frac{C_c}{C_{rus}}$$

where:

R = Concentration ratio;

C_c = Concentration in parts per million, by weight, of sediment in water entering the canal headgates;

C_{rus} = Concentration in parts per million, by weight, of sediment in the river water upstream from the canal headworks.

Thus, lower values of the ratio indicate a more satisfactory sediment exclusion device.

To develop a satisfactory set of vanes for the standard test discharge, it was assumed that for a given number of vanes having an established cross section, a satisfactory vane spacing for a given vane angle, location, length, and elevation would still be satisfactory if one of the other factors was varied. Accepting this assumption for all the variables involved allowed the following test plan to be adopted:

1. Determine satisfactory vane spacing.
2. Determine satisfactory vane angle.
3. Determine satisfactory placement of vanes with respect to canal headworks.
4. Determine satisfactory vane length.

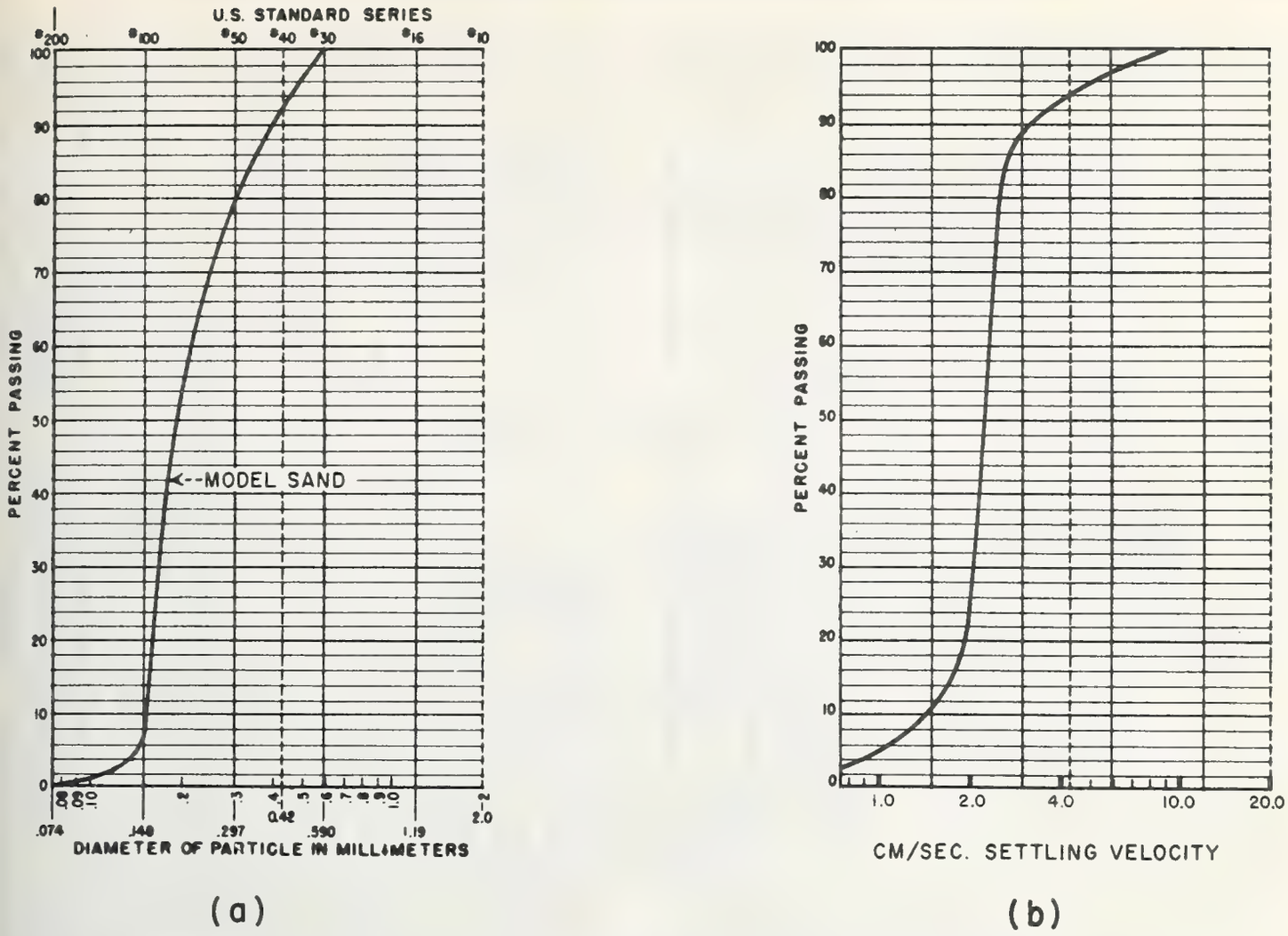


FIGURE 5. — *A*, Sand gradation analysis; and *B*, settling velocities in hydraulic model study.

5. Determine satisfactory vane elevation or vane depth.
6. Determine effect of the number of vanes.
7. Determine effect of vane cross section.

A graphical method of correlation analysis presented by Ezekiel[3] was used in analyzing results. In this method, a number of variables, such as the concentration ratio, any of the vane variables, concentration of total sediment moving in the river, and concentration of sediment moving near the headworks, are considered in the evaluation of a particular arrangement. The concentration ratios are first plotted as a function of the variable of immediate interest; for example, depth of vane (fig. 6). All points on the plot are numbered for future reference. The best average straight line is fitted to the points in graph 1, solid line. Deviations of points 1, 2, 3, and 4 from the fitted line (deviation 1) are then plotted as a function of the next most important variable; in this example, concentration of sediment moving in the river, C_{rus}. From this plotting, a new curve is fitted to the points (graph 2 of fig. 6), which is a correction curve to indicate the effect of the river concentration on the concentration ratio. Deviations from graph 2 (for example, deviation 2) are then plotted as a function of the variables of next importance; in this example, concentration of sediment in the sluiceway is C_s. From the resulting correction curve, C_s is shown to be of minor importance. This process is continued until all the independent variables desired have been introduced.

In the example, the concentration of sediment in the sluice was the last variable considered. A second approximation of the relation of the concentration ratio to the depth of vane may be determined by plotting the deviations from the last correction curve as deviations from the first approximation curve. A curve is fitted to the new points, and, if considerable change results or if a large number of points are available, it may be desirable to repeat the entire evaluation process.

Although a straight line of best fit can

[3] EZEKIEL, MORDECAI. METHODS OF CORRELATION ANALYSIS. Ed. 2. 1941. John Wiley and Sons, Inc.

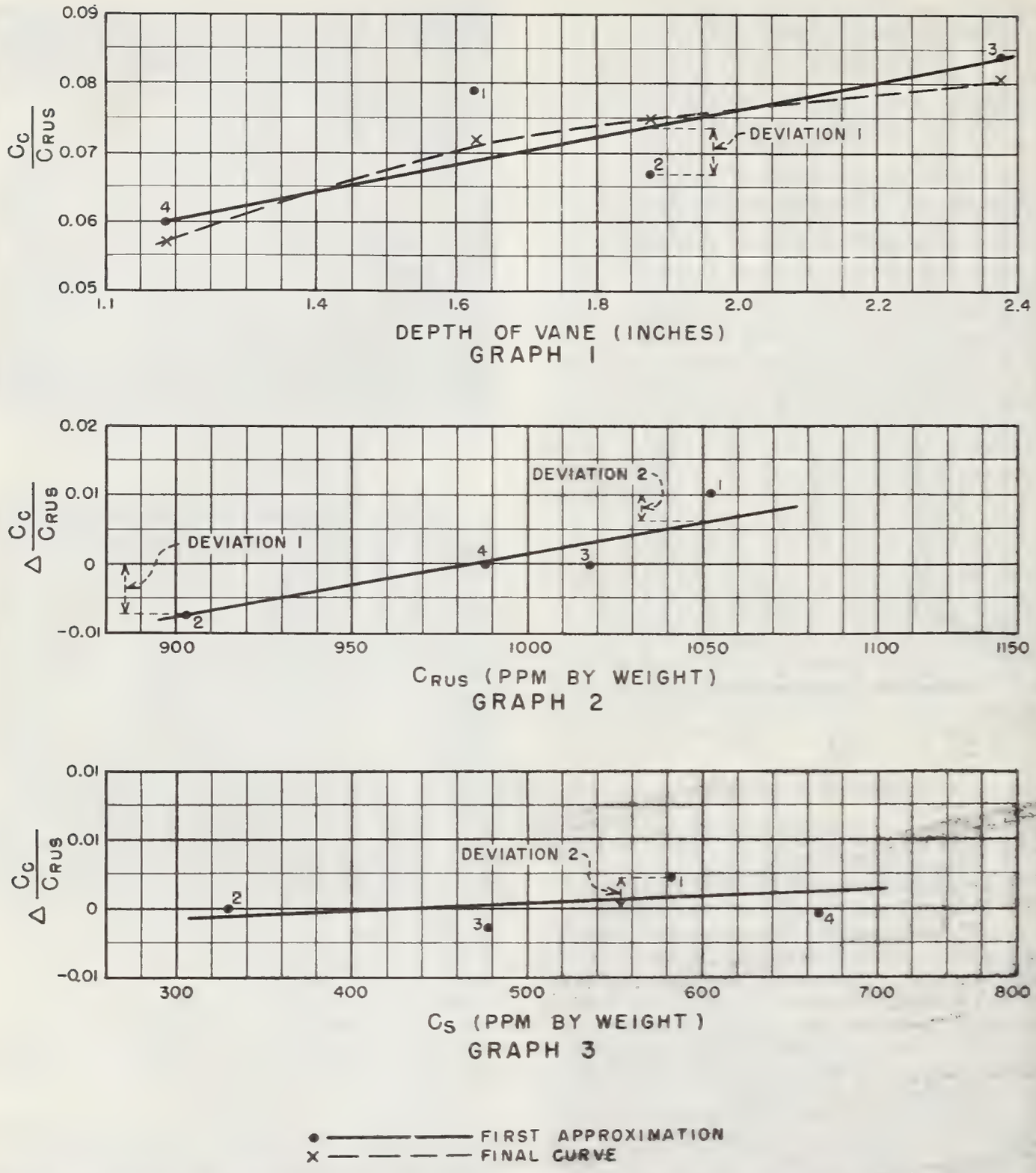


FIGURE 6. — Example of graphical method used for analysis of depth of vane, concentration of sediment in the river water upstream from canal headworks, and concentration of sediment in sluiceway.

usually be drawn to fit the points, this method is not limited to straight lines. If only a few points are involved a straight-line curve is easy and rapid to use. However, if too few points are involved results may be inconclusive. As only a limited number of points were available for these analyses (usually three or four), the conclusions drawn are necessarily limited.

Control Tests

Five tests were conducted without vanes but with a 160-foot by 40-foot 3-inch slab, at eleva-

tion 4,661.0, near the canal headworks on which the bottom vanes were later constructed. The standard discharges were used in the control tests and the river discharge was passed through all the river gates, which were opened equally. The duration of the five control tests averaged approximately 6 hours, and the control tests were spaced throughout the overall testing period. The control tests are tests 4 and 19 in the bottom guide vane series and tests 20, 24, and 36 in the surface guide vane series. Results are shown in table 1.

TABLE 1.—*Results of control tests with slab and without vanes for concentration of sediment*

Test No.	Hours test was conducted	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio $\frac{Cc}{C_{rus}}$
	Number	*P.p.m.*	*P.p.m.*	
4	7.0	1,601	385	4.16
19	4.4	1,635	345	4.74
20	8.4	1,589	1,331	1.20
24	4.1	886	835	1.06
36	5.0	622	861	.72

The average concentration ratio obtained from the control tests was 2.38. This value is therefore the datum used to determine the improvement resulting from the various guide vane arrangements. In the control tests, sediment deposits in the canal decreased the effective cross section of the canal and resulted in a gradual decrease in discharge as the test progressed. The average decrease in discharge was approximately 34 percent. When the vanes were in place, no appreciable decrease in canal discharge occurred.

Tests Conducted with Bottom Vanes

Bottom Vane Spacing

Four tests were made to determine a satisfactory vane spacing for tests using bottom vanes. These are shown in table 2 as tests 1, 2, 3, and 5 (test 4 was a control test). Vane arrangements for the tests are shown in figure 7, and the test data are summarized in table 2.

TABLE 2.—*Results of tests to determine satisfactory vane spacing with bottom vanes*

Test No.	Hours test was conducted	Vane spacing	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
1	26.2	16–8	41	233	0.176
2	51.2	12–0	15	161	.093
3	31.2	26–0	34	349	.097
5	31.2	20–0	44	262	.168

For these tests the four vanes used were 50 feet long, their top elevation was at 4,665.0 feet, and they were placed at an angle of 40° to the direction of flow with the downstream end of the downstream vane on the canal headworks centerline.

In these tests the concentration ratio was considerably improved from the 2.38 average of the control tests. A multiple correlation of the data indicated the spacing of 26 feet on centers in test 3 to be most satisfactory. For this spacing the concentration ratio was reduced to 0.097 for 349 p.p.m. sediment concentration in the river. The 26-foot spacing was used in all following tests.

Although the value 0.093 in table 2 appears to be more satisfactory than the value 0.097, the multiple correlation methods indicates 0.097 to be better. This results from a number of variables being involved in the correlation. For instance, although the concentration in the river was 349 p.p.m. for test 3, it was only 161 p.p.m. for test 2. However, even the difference between 0.093 and 0.176, the highest concentration ratio obtained in this test series, is not great when compared to the control concentration ratio of 2.38.

Angle Between Bottom Vanes and Direction of Flow

Tests 3, 6, and 8 were utilized to determine a satisfactory angle between the vanes and the direction of flow. The standard test discharge was set, and the multiple correlation method was used for analyzing results. For all three tests the vane length was 50 feet, vane spacing was 26 feet on centers, the tip of the downstream vane was on the canal centerline, and vane top elevation was 4,665.0 feet (fig. 7). No noticeable decrease in discharge occurred in the canal for these tests. Results of the tests are shown in table 3.

TABLE 3.—*Results of tests to determine a satisfactory angle between the vanes and direction of flow*

Test No.	Hours test was conducted	Vane angle to riverflow	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number		*P.p.m.*	*P.p.m.*	
3	31.2	40°	34	349	0.097
6	29.0	35°	65	367	.177
8	51.4	45°	33	310	.106

In the previous series of tests the concentration ratio was not sensitive to vane spacing; in these tests the concentration ratio was not overly sensitive to the angle at which the vanes were placed in the river. However, from a multiple correlation analysis of the data, the 45° angle was considered to be most satisfactory. All angles tested indicated considerable improvement in the concentration ratio as compared to the ratio obtained when no vanes were

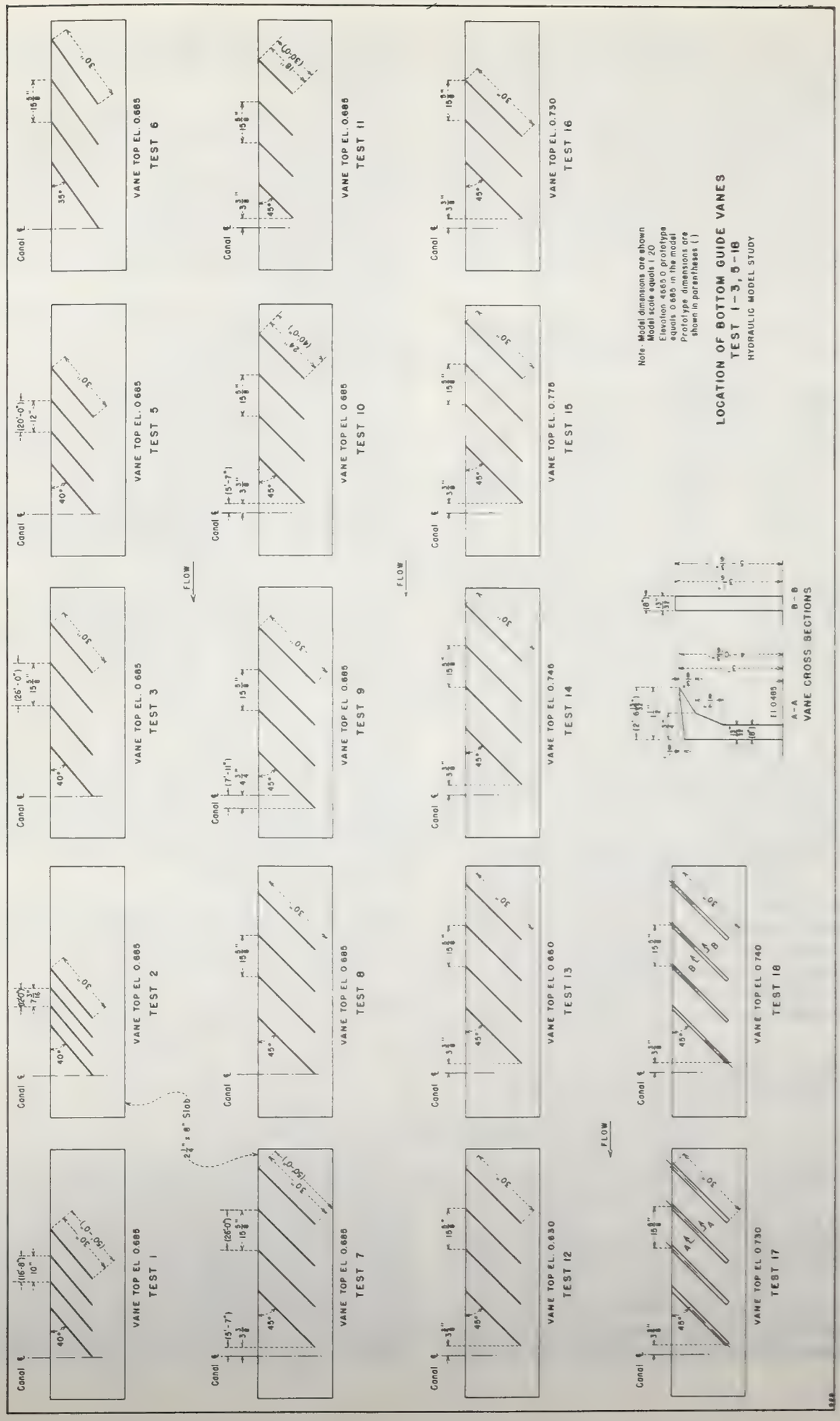


FIGURE 7. — Location of bottom guide vanes for tests 1 to 3 and 5 to 18.

installed. The 45° angle indicated an improvement in $\frac{C_c}{C_{rus}}$ from 2.38 to 0.106.

Bottom Vane Location

Tests 7, 8, and 9 were utilized to determine a satisfactory placement or location of vanes with respect to the canal headworks. The standard test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used, and the multiple correlation method was used in analyzing results. For all tests, the vane length was 50 feet, vane spacing 26 feet, vane elevation 4,665.0 feet, and the angle of the vane with the direction of flow was 45°. Test results are summarized in table 4.

TABLE 4.—*Results of tests to determine satisfactory placement of vanes with respect to canal headworks*

Test	Hours test was conducted	Tip location of downstream vane [1]	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number		*P.p.m.*	*P.p.m.*	
7....	26.8	5 feet, 7 inches upstream from canal (centerline).	22	467	0.047
8....	51.4	At canal (centerline).	33	310	.106
9....	27.9	7 feet, 11 inches downstream from canal (centerline).	28	380	.074

[1]See figure 7.

Visual observations of trial locations indicated that placing the vanes either farther upstream or downstream from the canal headworks would reduce the efficiency of the vanes. The multiple correlation analysis of the three tests indicated that placing the vanes 5 feet 7 inches upstream from the canal centerline was the most satisfactory arrangement.

Bottom Vane Length

Tests 7, 10, and 11 were used to determine a satisfactory vane length. The standard test discharge was set and the following conditions were constant for the three tests: vane spacing 26 feet on centers, vane elevation 4,665.0 feet, angle of vane with direction of flow 45°, and the tip of the downstream vane 5 feet 7 inches upstream from the centerline of the canal headworks. Table 5 summarizes results of these tests.

Plots and analysis of data indicated the 50-foot vane length to be most satisfactory. The results showed a considerable improvement over the average concentration ratio of 2.38 with no vanes in place.

TABLE 5.—*Results of tests to determine a satisfactory bottom vane length*

Test No.	Hours test was conducted	Vane length	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
7....	26.8	50	22	467	0.047
10....	30.6	40	16	303	.053
11....	29.8	30	34	333	.102

Vane Top Elevation

Tests 7, 12, 13, 14, and 15 were utilized to establish a satisfactory vane top elevation for the test discharge. For this series of tests, vane length was 50 feet, vane spacing was 26 feet on centers, angle of vane with direction of flow was 45°, and the tip of the downstream vane was placed 5 feet 7 inches upstream from the centerline of the canal headworks. Four vanes were used in all tests. Table 6 summarizes results of these tests.

TABLE 6.—*Results of tests to determine a satisfactory top elevation, bottom vane*

Test No.	Hours test was conducted	Vane top elevation	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
7....	26.8	4,665.0	22	467	0.047
12....	29.0	4,663.9	90	362	.249
13....	29.5	4,664.5	26	175	.149
14....	49.0	4,666.2	4	319	.013
15....	23.9	4,666.8	41	456	.090

The vanes installed for test 12 appeared to be too low and allowed considerable sediment to pass over them. An average decrease in discharge of approximately 5 percent occurred in the canal during test 12. No significant decrease in discharge occurred during the other tests of this series.

Analyses of the data indicated the most satisfactory surface elevation to be between 4,665.9 and 4,666.2 feet. The elevation selected as most satisfactory, after these and additional test data were analyzed, was 4,666.1 feet. However, in some succeeding tests the value 4,665.9 was used.

Number of Vanes

Test 16 was used to establish whether fewer than four vanes would produce sufficiently strong secondary currents to reduce sediment intake into the canal.

Three vanes were tested in a manner similar to that for four vanes. The 50-foot-long vanes were placed with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks. Vane top elevation was 4,665.9; the vanes were 26 feet on centers and placed at an angle of 45° with the direction of flow. The standard discharges were set, and tests of 18.5 and 7.0 hours were conducted. The canal discharge remained constant during the tests, and the resulting average concentration ratio was 0.067.

From visual observations and comparison of tests 16 and 14, it was concluded that four vanes produced a more satisfactory concentration ratio than three vanes.

Effect of Vane Cross Section

Tests 17 and 18 were utilized to determine the effect of vane cross section on the concentration ratio. In previous tests, the vanes in the model were constructed of sheet metal, equivalent to a thickness of approximately 1 inch in a prototype structure 20 times as large as the model. A prototype vane would be somewhat thicker, particularly a concrete vane, and tests were required to evaluate the effect of vane thickness. Vanes 8 inches thick were therefore investigated. Figure 7 shows cross sections of the vanes used in these tests.

Four 50-foot-long vanes were used in these tests, spaced 26 feet on centers, placed at an angle of 45° with the direction of flow, and with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks centerline. Vane top elevation for test 17 was 4,665.9 feet and for test 18 was 4,666.1 feet. The test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used for the tests.

In test 17, a set of four bottom vanes with sharp-edged lips extending 2 feet $6\frac{13}{32}$ inches upstream, shown in figure 7, were tested during runs of 18.7, 6.2, 17.0, and 6.9 hours. The canal discharge remained constant, but no significant improvement was shown over vanes made with a rectangular cross section. The average concentration ratio was 0.110.

Rectangular vanes 8 inches thick (fig. 7) were installed for test 18, and runs of 16.9 and 6.7 hours were conducted.

The canal discharge remained constant during both runs, and the resulting average concentration ratio was 0.094. The rectangular vane cross section used for this test appeared to be more satisfactory than the cross section used in test 17 and would certainly be easier to construct in a field installation.

Photographs of the sediment deposits that resulted in the canal when no control was used and when the vanes of test 18 were used are shown in figure 8.

Summary of Bottom Vane Tests

From these tests it was concluded that bottom vanes are effective in reducing heavy sediment intake into a canal supplied by water diverted from a large river. The most efficient bottom vanes developed were a group of four 50-foot-long vanes installed upstream from the intake at an angle of 45° to the direction of flow. The vanes were spaced 26 feet on centers, the downstream tip of the downstream vane was located 5 feet 7 inches upstream from the canal headworks centerline and vane top elevation

(a)
Without Bottom Guide Vanes

(b)
With Bottom Guide Vanes

FIGURE 8. — Sediment deposits in canal (*a*) without bottom guide vanes and (*b*) with bottom vanes in hydraulic model study.

was at 4,666.1. This arrangement reduced the concentration ratio for the test discharge from 2.38, the average for the five control tests with no vanes in place, to less than 0.1. This ratio reduction means, in effect, that the vanes allowed only 1/23 of the usual amount of heavy sediment to enter the canal. Tests conducted during the San Acacia model study [4] indicated that the vanes were also of benefit when the ratio of river to canal discharge was varied.

If the standard test discharge used in these tests is only considered, the dimensions of the variables, location, spacing, angle, length, depth, number, and cross section were not critical with respect to performance. Minor changes in the dimensions tested could be made, therefore, without changing performance significantly. Surface vanes were next investigated to determine whether they were more or less efficient than bottom vanes.

Tests with Surface Vanes

Surface Vane Spacing

Tests 21, 22, 23, and 25 were used to determine the effect of spacing of surface vanes (tests 20 and 24 were control tests). Vane arrangements for the tests are shown in figure 9 and the test data are given in table 7.

TABLE 7.—*Results of tests to determine satisfactory vane spacing for surface vanes*

Test No.	Hours test was conducted	Vane spacing	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{C_c}{C_{rus}}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
21....	29.9	16–8	59	944	0.062
22....	4.8	12–0	113	953	.119
23....	5.8	26–0	87	917	.095
25....	3.0	20–0	118	1,030	.114

Four vanes 50 feet long and 2 feet 8½ inches deep were placed with their bottoms at elevation 4,667.68; vane tops were at the normal water surface. The vanes were placed at an angle of 140° (measured from the same reference as the bottom vanes), with the downstream end of the downstream vane at the canal headworks centerline. In this test and all tests conducted with surface vanes, the vane thickness was 8 inches. The vane spacing was varied in each test as indicated in column 3 of table 7.

The test ratios all showed considerable improvement over the 2.38 average concentration ratio of the control tests and indicated that the concentration ratio was not greatly affected by the spacing of the vanes. However, a multiple correlation of the data showed that a spacing of approximately 18 feet 4 inches would be the most efficient for these vanes. From this series of tests it was concluded that surface vanes could be used effectively to reduce the sediment entering a canal headworks.

[4] See footnote 2.

Surface Vane Angle

In the bottom vane tests, the vanes were installed to guide bottom water away from the right bank into the river. When surface vanes are used to produce the same flow conditions they must be pointed toward the bank (looking downstream). In this position the vanes create secondary currents that move the bottom water away from the riverbank into the river. This action helps to exclude sediment from the headworks (fig. 2). Tests 23, 26, 27, and 28 were used to test various vane angles.

For all four tests the vanes were 50 feet long, vane spacing was 26 feet on centers, the downstream tip of the downstream vane was at the canal centerline, and the vane was 2 feet 8½ inches deep, with bottom elevation at 4,667.68 feet (fig. 9). Tests were made using the standard test discharges, and the multiple correlation method was used for analyzing results. Results of the tests are shown in table 8.

TABLE 8.—*Results of tests to determine satisfactory surface vane angle*

Test No.	Hours test was conducted	Vane angle	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{C_c}{C_{rus}}$
	Number		*P.p.m.*	*P.p.m.*	
23....	5.8	140°	87	917	0.095
26....	4.8	145°	97	875	.111
27....	3.0	135°	76	936	.081
28....	5.0	130°	48	1,007	.047

Although the concentration ratio was not very sensitive to the angle at which the vanes were placed, a correlation analysis indicated the 130° angle to be most satisfactory.

Surface Vane Location

Tests 27, 29, and 30 were used to test surface vane placement (location). For all tests, the vane length was 50 feet, vane spacing was 26 feet on centers, angle of the vane was 135°, vane depth was 2 feet 8½ inches, and vane bottom elevation was 4,667.68 feet (fig. 9). The standard test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used, and the multiple correlation method was used in analyzing data. Test results are summarized in table 9.

These three tests indicated that the vanes were remarkably insensitive to exact location. Concentration ratios were very similar for all three tests. However, visual observations indicated

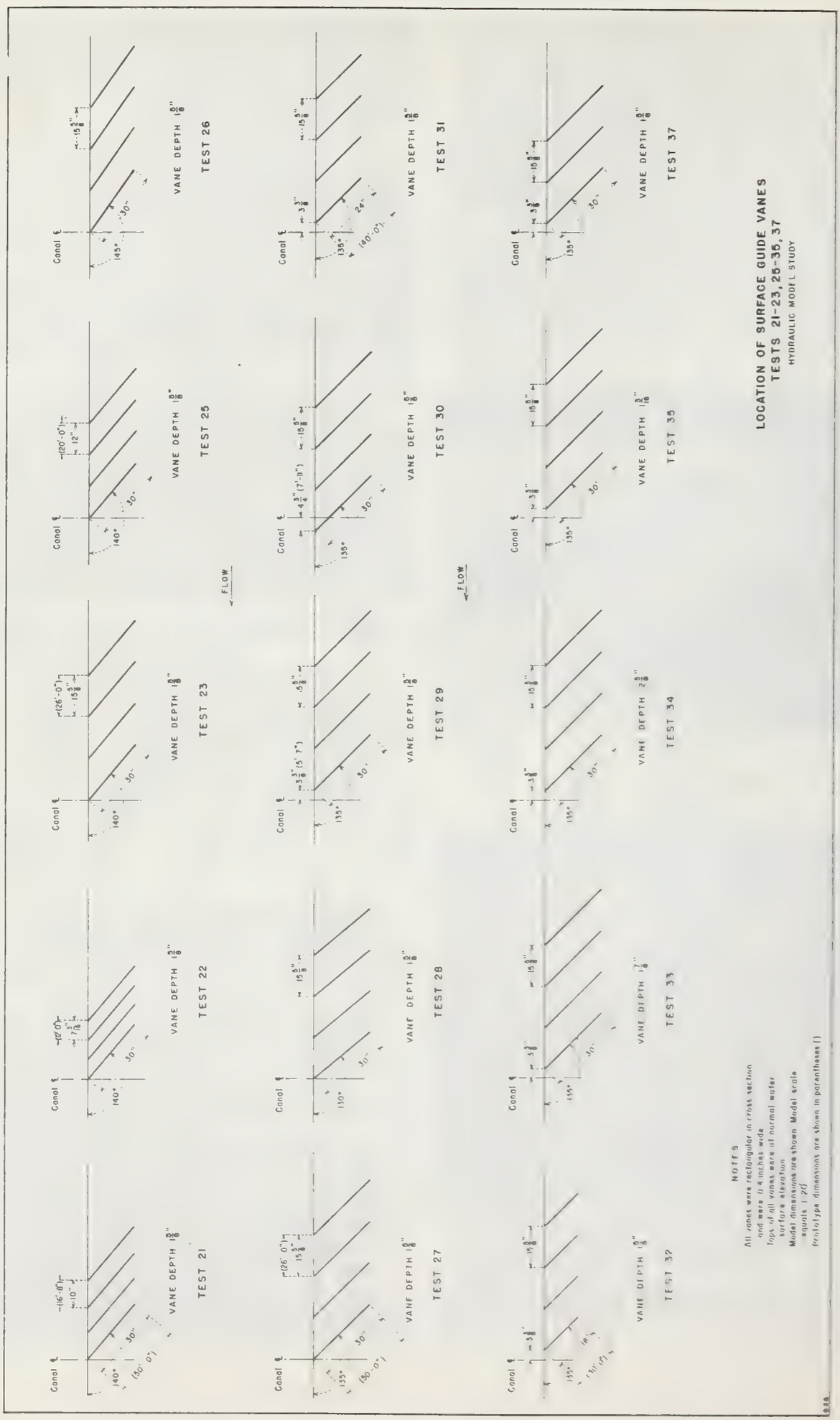


FIGURE 9. — Location of surface guide vanes for tests 21 to 23, 25 to 35, and 37.

that moving the surface vanes farther upstream or downstream would have reduced their efficiency.

TABLE 9.—*Results of tests to determine satisfactory placement of surface vanes with respect to canal headworks*

Test No.	Hours test was conducted	Tip location of downstream vane	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{Crus}$
	Number		*P.p.m.*	*P.p.m.*	
27...	3	At canal (centerline).	76	936	0.081
29...	5	5 feet, 7 inches upstream from canal (centerline).	83	1,052	.079
30...	5	7 feet, 11 inches downstream from canal (centerline).	62	824	.075

Surface Vane Lengths

Tests 29, 31, and 32 were used to show the effect of varying the length of surface vanes on their efficiency in controlling sediment movements. The standard test discharges were used and the following conditions were maintained constant: Vane spacing was 26 feet on centers, angle of vane was 135°, the tip of the downstream vane was placed 5 feet 7 inches upstream from the canal centerline, the vanes were 2 feet 8½ inches deep, and vane bottom elevation was 4667.68 feet (fig. 9). Table 10 summarizes results of these tests.

TABLE 10.—*Results of tests to determine a satisfactory surface vane length*

Test No.	Hours test was conducted	Vane length	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{Crus}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
29....	5	50	83	1,052	0.079
31....	5	40	57	861	.066
32....	5	30	62	863	.072

The analysis indicated that a surface vane longer than 40 feet did not improve the concentration ratio, and that a vane less than 40 feet long was slightly less efficient.

Surface Vane Height

Tests 29, 33, 34, and 35 were utilized to establish a satisfactory height of surface vanes. For this series of tests the vane length was 50 feet, vane spacing was 26 feet on centers, and the angle of the vanes was 135°. Four vanes were used, and the tip of the downstream vane was placed 5 feet 7 inches upstream from the centerline of the canal headworks. Table 11 summarizes results of these tests.

TABLE 11.—*Results of tests to determine a satisfactory surface vane height*

Test No.	Hours test was conducted	Vane height	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{Crus}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
29....	5	2— 8½	83	1,052	0.079
33....	5	3— 1½	61	903	.067
34....	5	3—11½	84	1,018	.084
35....	5	1—11¾	59	988	.060

In all cases, the vanes were effective, but analysis indicated the most effective depth to be 1 foot 11¾ inches.

Number of Surface Vanes

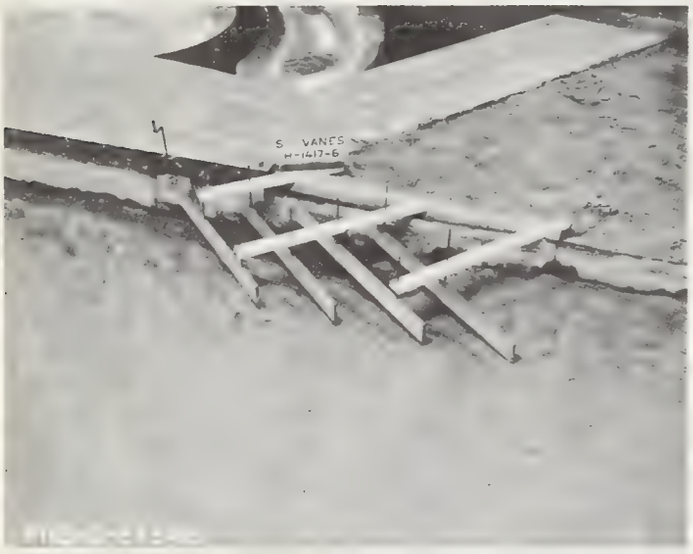
Test 37 was conducted to establish the effect of fewer than four vanes (test 36 was a control test). Three 50-foot-long vanes were placed with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks. Vane depth was 2 feet 8½ inches, spacing was 26 feet on centers, and vanes were placed at an angle of 135°. The standard discharges were tested and the resulting concentration ratio was 0.074, almost identical with the ratio 0.079 of test 29 in which four vanes were used.

Summary of Surface Vane Tests

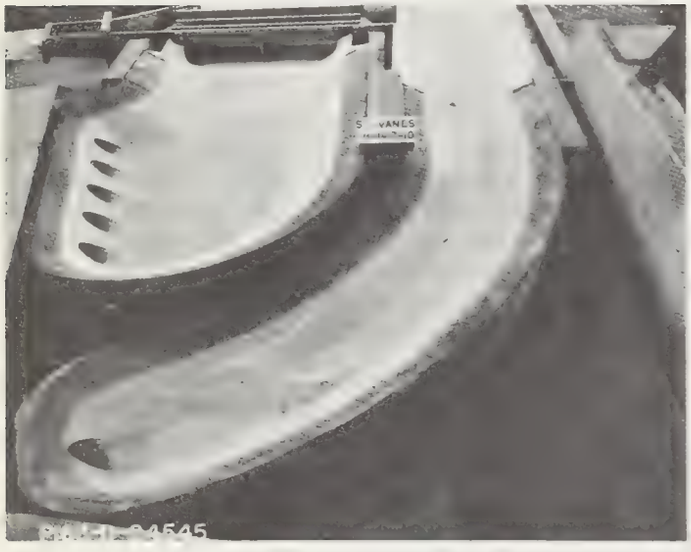
From the tests with surface vanes, it was concluded that surface vanes are effective in reducing heavy sediment intake into a canal supplied with water diverted from a large river and are about as efficient as bottom vanes. The dimensions of the vane variables, location, spacing, height, angle, length, and number were not critical with respect to the performance of the set of vanes. Installed in the same relative position as the bottom vanes but angled so as to divert top water into the canal headworks (bottom vanes are angled so as to divert bottom water away from the headworks) the surface vanes produced approximately equivalent results to bottom vanes and reduced the concentration ratio for the test discharge from 2.38, the average for the control tests, to less than 0.1. All tests on the surface vanes were conducted at standard discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second diverted to the canal.

Photographs showing the surface vanes in

place for test 21 and the resulting deposits in the canal following the tests are shown in figure 10.



(a)
Surface Vanes in Place for Test 21



(b)
Sediment Deposits in Canal After Test 21

FIGURE 10. — Surface vanes in place (*a*) and sediment deposits in canal (*b*) after test 21.

Comparison of Vanes and Discussion

Both surface and bottom guide vanes reduced the concentration ratio $\frac{C_c}{C_{rus}}$ (concentration of sediment entering the canal to concentration of sediment in the river upstream) from 2.38, the average of the control tests, to less than 0.1. In other words, the sediment entering the headworks was only 1/23 the amount that entered when no vanes were used. A comparison of the vane variables that produced the most satisfactory results for the bottom and surface vanes is shown in table 12. Although the results of tests with both types of vanes showed that the vanes were not overly sensitive to the variables tested (within the test range), the surface vanes appeared to be the least sensitive to location, length, and numbers of vanes. The angle at which the bottom vanes worked best was 45°. Also, the ratio, height of vane divided by the water depth above the slab, may be less for the surface vanes. The lesser height of the surface vanes appears to result in less total projected vane area in the flow prism. However, as some sediment was always present on the supporting slab when the bottom vanes were in place (fig. 11) the y/d ratio and projected area shown for the bottom vanes in table 12 is no doubt too large. Few generalizations can be made from these tests because only one arrangement of canal intake and only one discharge were tested. The tests demonstrated, however, that satisfactory vane arrangements can be developed by the use of the testing and analysis principles discussed.

The most effective set of bottom vanes tested consisted of four 50-foot-long vanes installed upstream from the canal headworks. The vanes were placed along the right bank of the river model at an angle of 45° to the direction of flow. Vane spacing was 26 feet on centers, vane top elevation was 4,666.1 feet, and the downstream tip of the downstream vane was located 5 feet 7 inches upstream from the canal headworks centerline. The bottom vanes tested are shown in figure 7.

The most effective set of surface vanes, indicated by the tests, included either three or four vanes 40 to 50 feet long placed near the canal headworks (table 12). The vanes were installed along the right bank of the river model at an angle of 130° (same reference datum as bottom vanes). Vane spacing was 18 feet 4 inches on centers and vane height was 1 foot 11¾ inches. The surface vanes tested are shown in figure 9.

Both bottom and surface vanes were found to be extremely valuable in helping to gain control of heavy sediments by creating localized secondary currents to reduce the sediment intake into a canal. Consideration should be given to their use where flow conditions are similar to those tested in this study. For example, where a relatively small discharge is being diverted from a relatively large flow and it is desired that the small discharge have a relatively light sediment load, either bottom or surface vanes may be used with the dimensions (or propor-

TABLE 12.—*Optimum dimensions of variables of bottom and surface vanes*

Vanes	Best spacing		Best angle tested	Best placement tested	Best vane length tested	Best y/d ratio tested[1]	Effect on concentration ratio from reducing number of vanes from 4 to 3	Total vane surface in flow above slab at best condition
	Feet	*Inches*			*Feet*			*Sq. ft.*
Bottom.....	26	0	45°	Tip of downstream vane 5 ft. 7 in. upstream from canal (centerline)	50	0.544	Increases...	83.3
No. 7 Surface	18	4	130°	Tip of downstream vane from 7 feet, 11 inches downstream from canal (centerline) to 5 feet, 7 inches upstream from canal (centerline).	30–50	.257	No significant change.	33.3

[1] y = height of vane; d = depth of water above slab.

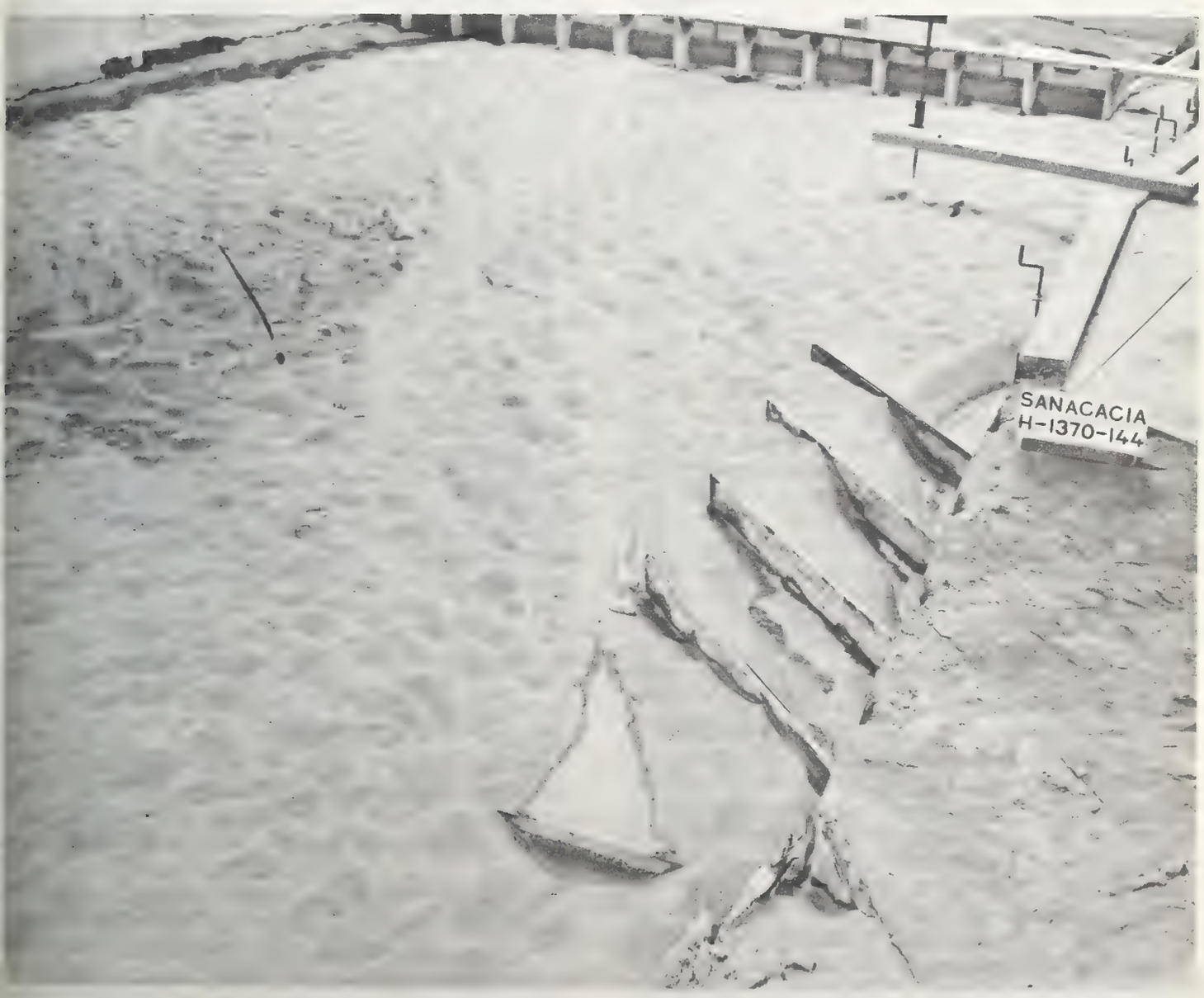


FIGURE 11.—Bottom vanes after test 18, showing sediment deposits partially covering vane area.

tional dimensions) given in this paper. Further investigation in a model should be made, however, if discharges are significantly different than those tested.

Further research should be conducted to determine the general performance of vanes in confined spaces and their possible use on increasing sediment loads in canal sluiceways. The effect of varying discharges should also be investigated.

SOME ASPECTS OF THE PROBLEM OF SCOUR AT BRIDGE CROSSINGS

[Paper No. 34]

By EMMETT M. LAURSEN, *professor and head, Department of Civil Engineering, The University of Arizona*

General Aspects of the Problem

As an agent of geological change, a stream is ever-changing. As an agent of sediment transportation, a stream is always tending toward equilibrium. However, as conditions change (such as the flow of the stream, the sediment supply to the stream, and the overall slope available), the equilibrium toward which a stream is tending will vary. In a natural stream the flow will contract and expand, and the streambed will be molded so that the capacity to transport sediment at every point is equal to the supply of sediment to that point. If a road crosses an alluvial valley, the embankment and the structures interfere with the natural pattern of flow. Generally there is a severe contraction of the flow and, consequently, severe remolding of the streambed — scour around the piers and abutments.

Ideally, the analysis of scour should start with a determination of the flow pattern and the boundary stresses, then proceed to a determination of the sediment transportation from point to point as a function of the bed material characteristics and the flow. The rate of scour then would be the difference between the transport capacity and the supply. As the scour progresses the boundary would change, changing the flow pattern and the stresses on the boundary. This would result in a change in capacity and supply from point to point and, thus, a change in the rate of scour. Eventually a stable condition, or limit, would be attained.

Unfortunately, the boundary geometries encountered are such that the flow pattern and the boundary stresses cannot be specified, and, moreover, their relation to the rate of sediment transport is not yet well established. However, in a few simple cases conditions can be described reasonably well with a minimum of conflict of opinion. The more complex geometries can then be approximated from these simple cases. Laboratory experiments tend to verify the analyses, but field investigations are needed for confidence in usage.

Clear-Water Scour and Scour by a Sediment-Transporting Flow

So many of the past investigations of scour seem to have confused the case of clear-water scour and scour with a sediment supply; it is worth while to make the distinction between these two cases. The simplest case of scour is that of a long contraction in which the transition between the two widths and such niceties as dune formation are ignored. Given two widths, a depth of flow (and slopes to compensate for the resistance to flow) the velocity of flow can be so low that there is no movement of the sediment anywhere, no matter how small the sediment or how great the contraction. There is clear water flowing in both reaches under these conditions. The flow pattern is that of the fixed boundary and, if the flow is subcritical, the depth of flow in the contraction is less than that in the approach reach. Therefore, the velocity and the boundary shear in the contraction are greater than in the approach reach. If the depth of flow is kept constant in the approach channel and the rate of flow is increased, the critical tractive force will be attained first in the contraction. Any further increase in the rate of flow will result in a boundary stress that will cause movement in the contraction and, therefore, scour. The greater the flow, the greater must be the scour, since it will continue until the depth is increased and the velocity decreased so that the boundary stresses again are critical for movement.

It should be readily apparent that in clear-water scour, the velocity and the sediment size, together with the geometry of the location, are important factors in determining the depth of scour.

If the discharge is increased to such an extent that in the approach flow the boundary shear is much greater than the critical tractive force of the sediment, there will be a continuous supply of sediment into the contraction. If the rate of movement is high enough, the critical

tractive force of the boundary sediments will have little influence. The bed in the contraction will be scoured until the boundary stresses are able only to transport the material supplied to the contraction. Now if the geometry were to remain the same for a higher discharge, the velocities everywhere should be increased proportionately and the shears also should increase everywhere proportionately. Therefore, the sediment transport rate at every point should increase in the same proportion; there should be no difference in the depth of scour no matter what the velocity or, using the same argument, what the sediment size. Few qualifications are necessary for this argument to be a good approximation. The Froude number should not be changed so greatly as to result in a change of the geometry of the free surface. The mode of movement and, therefore, the relation between the flow and sediment transport should be the same in the approach and in the contraction, and the rate of movement should be sufficiently high that the critical tractive force can be ignored. These qualifications are usually well satisfied for rivers in flood, and they have not given difficulty in laboratory experiments.

It should be clear that there is a difference between clear-water scour and scour by sediment-transporting flow and that conclusions reached from experiments on either one cannot be simply transposed to apply to the other.

The Iowa Investigations

Beginning in 1948, a series of investigations of scour around bridge piers and abutments were conducted at the Iowa Institute of Hydraulic Research, sponsored by the Iowa State Highway Commission and the U.S. Bureau of Public Roads. The first investigations were qualitative studies of typical pier and abutment geometries; followed by studies of the effect of sediment size and of velocity (*8*).

The experimental results of the second phase of the investigation indicated the lack of effect of these two factors on the depth of scour. It was noticed in the course of this phase of the investigation that the flow pattern was entirely different from that of two-dimensional flow. The sediment approaching the scour hole came straight down the flume and dropped into the scour hole. The sediment movement and the flow outside of the scour hole did not seem to pay any attention to the obstruction. The flow approaching the scour hole ducked down into the scour hole and came out as a spiral roller.

The third phase of the Iowa experiments involved the embankment on the flood plain that forces the overbank flow back into the main channel. Again the same characteristics were noted and served eventually as the basis for an approximate analysis of the scour phenomena. The results have been published elsewhere and will be merely summarized here (*5, 6*). The analysis begins by extending the ideas of Straub for the long contraction (*10*). The definition sketch (fig. 1) shows an approach reach with

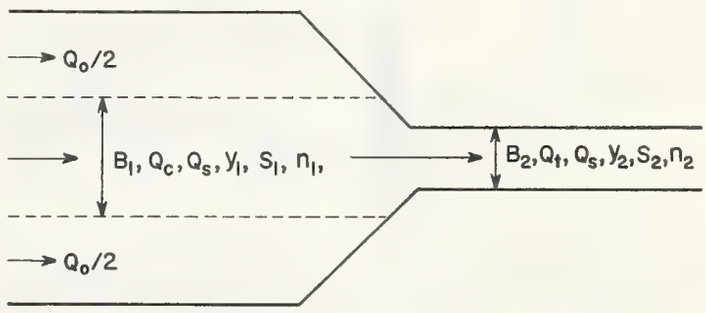


FIGURE 1. — Definition sketch of long contraction.

an overbank flow that does not transport any sediment and a contraction that constricts not only the flood-plain but the normal channel (see p. — for list of symbols). If the Manning formula to describe the flow in the two reaches and a total sediment load relation proposed by the writer (*4*) are used, the following expression for the depth ratio in the two reaches can be obtained:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{\frac{6}{7}} \left(\frac{B_1}{B_2}\right)^{\frac{6}{7}\,\frac{2+a}{3+a}} \left(\frac{n_2}{n_1}\right)^{\frac{6}{7}\,\frac{a}{3+a}} \qquad (1)$$

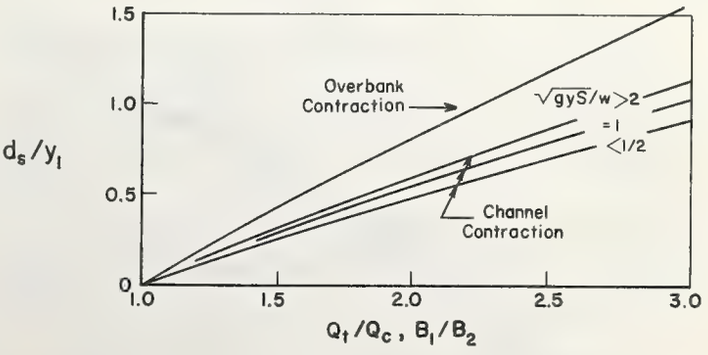


FIGURE 2. — Scour in a long contraction.

Figure 2 presents in graphical form the depth of scour as the difference between the two depths of flow as a function of a pure channel contraction or a pure overbank contraction, neglecting any effect of the n values, or kinetic terms. This solution for the long contraction can be extended to the overbank constricted by an embankment with the definitions shown in figure 3. Here the width of the approach channel is the length of the embankment plus 2.75

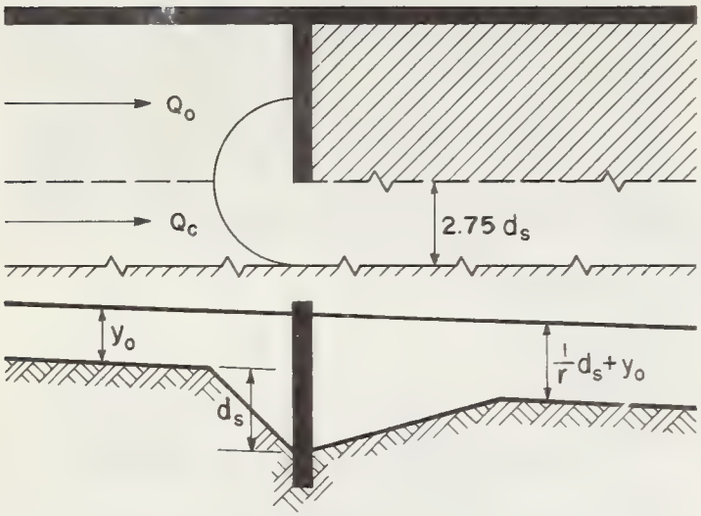


FIGURE 3. — Definition sketch of overbank constriction.

times the depth of scour at the abutment (i.e., the width of the scour hole), and the width of the contraction is 2.75 times the depth of scour at the abutment. The scour in the contraction is taken as a fraction $1/r$ of the scour at the abutment. Substitution of these definitions into the solution for the long contraction gives a relation for the depth of scour at the abutment of an overbank constriction.

$$\frac{Q_o}{Q_w}\frac{w}{{}^{o}\!/\!_{1}} = 2.75\,\frac{d_s}{y_o}\left[\left(\frac{1}{r}\frac{d_s}{y_o}+1\right)^{7/6}-1\right] \qquad (2)$$

Successive trials of w are made until w is approximately 2.75 d_s; Q_w is the flow in the width w. The results of the laboratory experiments are shown in figure 4 and a value of r of 4.1 gives rather good agreement between analysis and measurement. The points plot slightly above the line, but this is due to the fact that the flow on the overbank was actually greater than measured, because of crossflow in the approach. A similar relation can be obtained for the encroachment abutment.

$$\frac{l}{y_o} = 2.75\,\frac{d_s}{y_o}\left[\left(\frac{1}{r}\frac{d_s}{y_o}+1\right)^{1.70}-1\right] \qquad (3)$$

Good agreement with experiment is obtained with an r of 11.5, as shown in figure 5. The

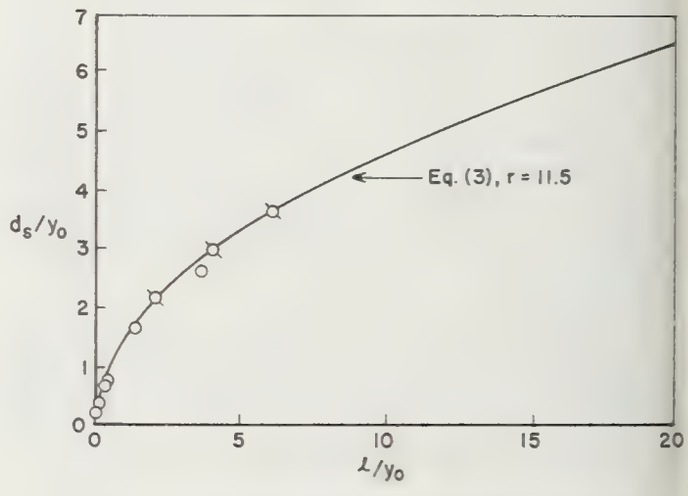


FIGURE 5. — Scour at an encroaching abutment.

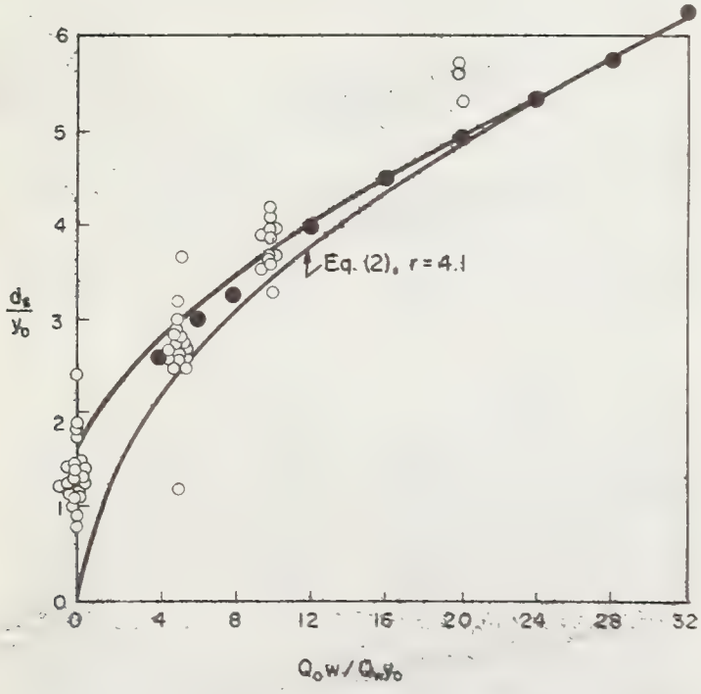


FIGURE 4. — Scour at an overbank constriction.

solution for the encroaching abutment can be adapted to the pier alined with the flow if the half width of the pier $b/2$ is substituted for the length of the encroaching embankment. In figure 6 the analytical relationship is compared to a design curve drawn conservatively on the

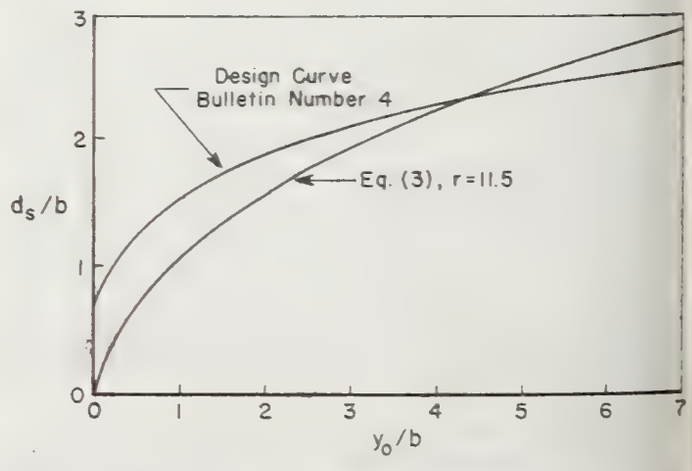


FIGURE 6. — Scour around basic bridge pier.

basis of the experiments with the piers. Geometry variations, such as a skewed crossing or pier, or variations in the mode of movement can be compensated for by correction factors.

A similar analysis has been performed for the case of clear-water scour (*7*). For the case of sediment-transporting flow, the conditions imposed were those of continuity of the water discharge and of the sediment transportation. For the case of clear-water flow, continuity for the discharge is again imposed, but, in place of the sediment-transport rate, the shear in the approach and in the contraction is specified. The shear is evaluated from the Manning formula and Strickler's definition of n. In the contraction the boundary shear is equated to the critical tractive force. The definition sketch for the long contraction is shown in figure 7. The

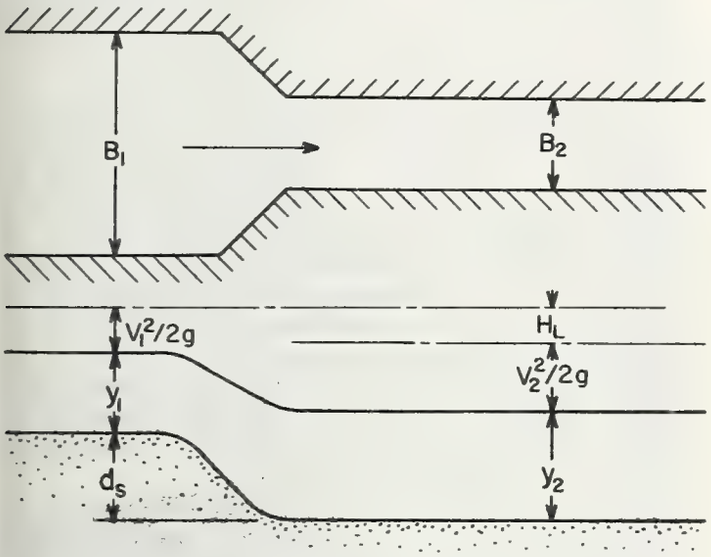


FIGURE 7. — Definition sketch of long contraction.

expression for depth of scour for this case of the long contraction can be obtained, considering the kinetic terms (the velocity heads and the loss in the transition) as:

$$\frac{d_s}{y_1}=\left(\frac{\tau_0'}{\tau_c}\right)^{3/7}\left(\frac{B_1}{B_2}\right)^{6/7}-1+1.87\,(1+K)\left(\frac{D}{y_1}\right)^{2/3}\left[\frac{(B_1/B_2)^{2/7}}{(\tau'_0/\tau_c)^{6/7}}-1\right]\left(\frac{\tau_0'}{\tau_c}\right) \qquad (4)$$

or neglecting the kinetic terms, a much simpler expression is obtained:

$$\frac{d_s}{y_1}=\left(\frac{\tau'_0}{\tau_c}\right)^{3/7}\left(\frac{B_1}{B_2}\right)^{6/7}-1 \qquad (5)$$

This latter relation is shown in figure 8. It is readily apparent that not only the geometry but the velocity of flow and the sediment size are important in determining the depth of scour. The solution for the long contraction can again be applied to the pier and abutment by defining the depth of scour in the long contraction as a fraction of that at the pier or abutment and the width of the contraction as the

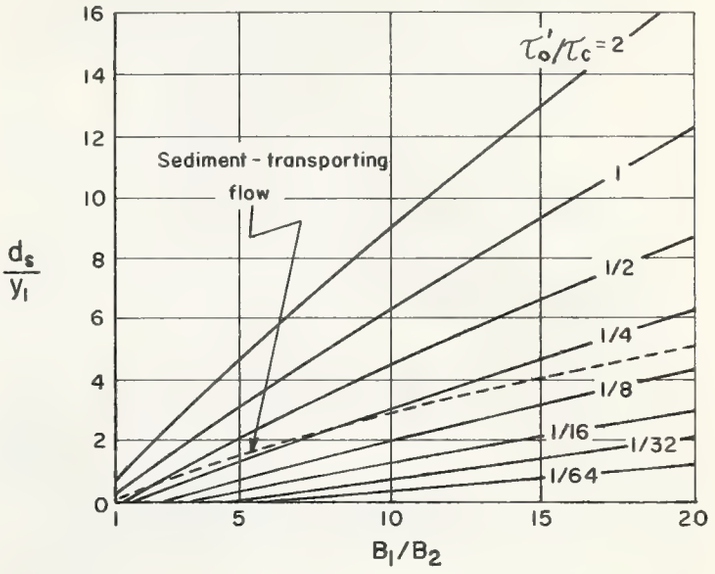


FIGURE 8. — Effect of shear ratio and width ratio on scour in a long contraction.

width of the scour hole. If the kinetic terms are neglected, the expression for the depth of scour for the abutment is:

$$\frac{l}{y_0}=2.75\,\frac{d_s}{y_0}\left[\frac{\left(\frac{1}{r}\frac{d_s}{y_0}+1\right)^{7/6}}{(\tau_0'/\tau_c)^{1/2}}-1\right] \qquad (6)$$

where $\tau_0'/\tau_c=V_0^2/120\,D^{2/3}y_0^{1/3}$.

In terms of pier geometry the half-width $b/2$ is substituted for the length of the embankment. The results are shown graphically in figures 9 and 10.

This approximate analysis of clear-water scour gives results that seem "right." Comparison with experiments from Poona (*3*), Chatou (*2*), and Fort Collins (*9*) are promising.

Implications of the Investigations

Several possibilities would seem to be implied, based on the results of the experiments and analyses. It would seem clear that the flow through a waterway opening is highly nonuniform, that dividing the total discharge by the nominal opening does not give a realistic "velocity" of flow which will result in a safe design. Unless the scour is so deep that scour holes overlap, the scour at the piers will not be affected by the overall contraction of the flow. The flow around the abutment is the flow interrupted by the embankment on the flood plain and

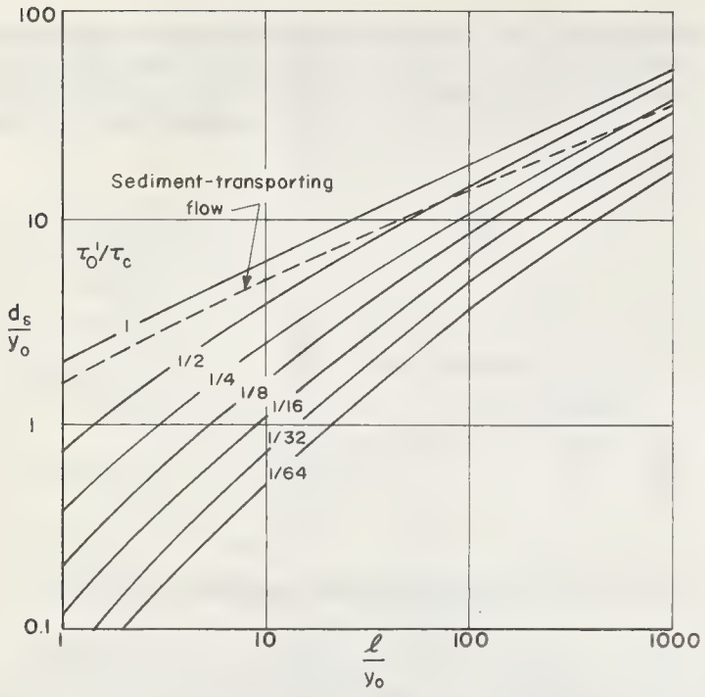


FIGURE 9. — Clear-water scour at an abutment.

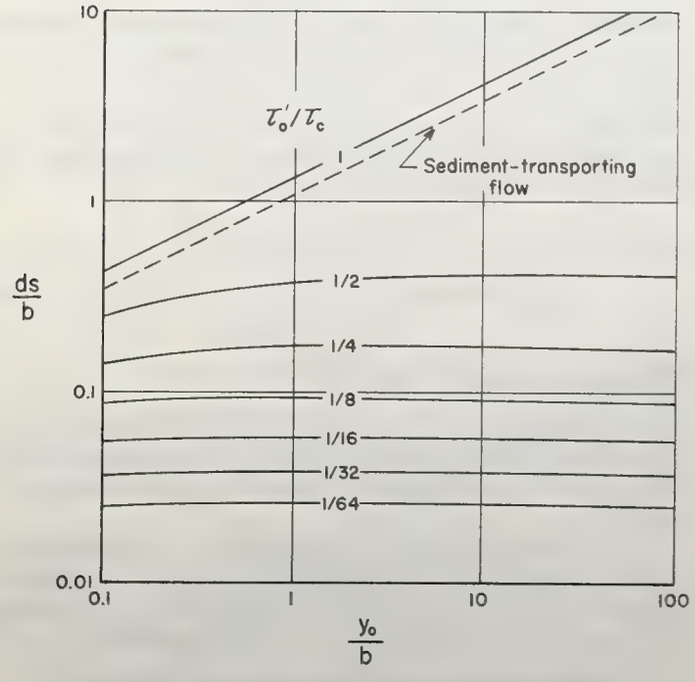


FIGURE 10. — Clear-water scour at a pier.

the length of the bridge itself is of small consequence. These conclusions are certainly to be taken as first approximations, because the geometry of a real crossing is much more complex than that of the schematic laboratory models. Moreover, abutments set back on the flood plain may be protected from scour because of the cohesive nature of the sediments. The typical layout with a cleared area upstream of the embankment, perhaps even a parallel ditch, tends to concentrate the overbank flow on the flood plain and, therefore, results in a different flow pattern from that of the laboratory. However, the concept that scour around bridge piers and abutments is a local phenomena should be considered in any criterion for waterway opening requirements.

Riprap and mattings have often been used in the past to protect against scour. The question has always been what size riprap should be used. The experiments and the analyses imply that riprap can be designed on the basis of clear-water scour and that the deeper the riprap is placed below the bed of the stream, the smaller it can be. An idea of the areal extent of the riprap can be gained from Schneible's experiments with a rigid mat extension of the pier placed below the bed of the stream.[1]

An ability to predict scour for a given geometry for different floods permits design from an "insurance" concept. There will be a certain depth of scour for a 50-year flood that has a certain chance of occurrence (one minus the chance of nonoccurrence) in the life of a bridge. The scour will be deeper for a 100-year flood, but there will be a smaller probability of occurrence. The possible loss weighted by the decrease in probability of occurrence can be balanced against the extra cost for rare floods, which can be either in the form of deeper foundations or riprap protection.

The scour in the long contraction is generally much less than a local scour around the piers or abutments. If the long contraction scour can be obtained by proper design of guide dikes upstream of the bridge, the cost of such dikes may be less than the cost of deeper foundations. Much shorter bridges are a possibility. A clue to the proper design of the guide dikes may be the constant-pressure transition curve (*1*).

Further Investigations Needed

The understanding of scour in the field is not yet at the point that predictions based on analyses and laboratory experiments can be used with complete confidence. What is needed is a variety of field measurements of scour to verify the relationships proposed or to point out deficiencies in those relationships. It is to be expected that the complexity of the field has not been adequately evaluated in the schematic laboratory and analytical models. The field measurements should be made in situations of simple geometry that can be correlated with the laboratory models and in situations of complex geometry that go beyond the schematic

[1] SCHNEIBLE, D. E. AN INVESTIGATION OF THE EFFECT OF BRIDGE-PIER SHAPE ON THE RELATIVE DEPTH OF SCOUR. Master's thesis. On file, library, State University of Iowa. 1951.

laboratory models. One might expect that as a result of these field measurements further experimental studies would be indicated. Judgment will always be an important factor. The floods to be expected are those of rare occurrence. The field conditions of the future can change drastically from those of today. However, it is better to have judgment related to these questions than directly to the question of how deep a bridge foundation should be placed.

List of Symbols

a ..... An exponent in an approximate sediment transport equation (*4*, *5*, *6*).

b ..... Width of a pier.

B ..... Width of flow section; subscript 1 refers to approach channel, subscript 2 refers to contracted channel.

d_s ..... Depth of scour measured from original stream bed.

D ..... Diameter of sediment particles.

K ..... Loss coefficient through constriction.

l ..... Effective length of constricting embankment.

n ..... Manning coefficient; subscript 1 refers to approach channel, subscript 2 refers to contracted channel.

Q ..... Rate of flow or discharge; subscript t refers to total flow, subscript c refers to flow in normal channel, subscript w to the flow in the channel over a width w measured out from the abutment.

r ..... Ratio between depth of scour at pier or abutment to depth of scour in fictitious long contraction.

w ..... Width of the fictitious long contraction equal to 2.75 d_s.

y ..... Depth of flow; subscript 1 refers to approach channel, subscript 2 refers to contracted channel, subscript 0 refers to depth of flow near pier or abutment.

τ_0 ..... Shear associated with sediment particle.

τ_c ..... Critical tractive force of sediment particle.

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DISCONTINUOUS DEPTH-DISCHARGE RELATIONS FOR SAND-CHANNEL STREAMS AND THEIR EFFECT ON SEDIMENT TRANSPORT

[Paper No. 35]

By David R. Dawdy, *U.S. Geological Survey, Menlo Park, Calif.*

Abstract

A discontinuity occurs in the depth-discharge relation of many sand-channel streams. For the higher part of the relation, after the discontinuity, Froude number and Chezy C resistance coefficient remain constant. The Chezy C approximately doubles, owing to the discontinuity. A concomitant increase on the order of eight times occurs in the transported sands. The effect of heavy loads of suspended fine material on these relations is discussed.

A discontinuity occurs in the depth-discharge relation of many sand-channel streams. For a given depth, there may be no unique relation with velocity. This is due to the fact that the form of bed roughness and, hence, resistance to flow are a function of fluid, sediment, and flow characteristics.

Laboratory studies (*10*) have been used to define the various regimes of flow in terms of configuration of the sand-bed channel. Those which apply to the field case are as follows:

Regime of flow	*Description of bed*
Lower regime:	
Ripple	Small, uniform sand waves, with little sediment movement.
Dunes	Much larger, more irregular sand waves, with a great deal of turbulence.
Upper regime:	
Plane bed	Dunes are smoothed out. Both the water surface and the bed are plane, with little turbulence.
Standing waves	Both the water surface and the bed are characterized by standing waves, often termed "sand waves."
Antidunes	The "sand waves" move upstream, until at some critical point they break, then re-form.

The discontinuity occurs between the dune regime and the plane bed regime. The roughness of the channel in the lower regime is about twice the roughness for upper regime. For upper regime, roughness may be considered essentially a grain roughness, whereas for lower regimes, there is both grain and form roughness (*8*).

The general approach to the determination of roughness is through the Chezy formula:

$$V = C\sqrt{RS}$$

which means that the velocity (V) at a cross-section varies as the square root of the hydraulic radius (R), if slope (S) and roughness (C) are constant. The usual adaptation to rigid channels of this relation is the well-known Manning's formula, where

$$C = \frac{1.5\,R^{1/6}}{n}$$

In this relation, velocity varies as the two-thirds power of hydraulic radius, if slope and Manning's n are constant. If slope can be considered as a constant with only statistical variation, a comparison of the variation of velocity with hydraulic radius can be used to study the variation of roughness.

Figure 1 shows such plots for four U.S.G.S. gaging stations on sand-channel streams. On each graph, the trend for the upper measurements is represented by a curve of relation. Each of these curves can be expressed as

$$V = kR^{1/2}$$

where k represents a constant in the relation. According to this relation, the Froude number remains constant, because

$$F = k/g^{1/2}$$

If the energy slope remains constant at a given stream site, each curve also represents a constant value of the Chezy roughness coefficient C for

$$C = k/S^{1/2}$$

contains only constant terms.

These stations are excellent examples of the non-uniqueness described by Brooks (*3*). This type of rating was described by Einstein and Chien in their discussion of Brooks, but they believed this type to occur rarely, if ever, in nature. It is now general knowledge that discontinuities are not rare in nature, but occur in many sand bed channels with "sufficient" slopes and depths (*7*). The stream gager in the field has long known that stage-discharge relations for many sand bed channels "went to Hell in a handbasket." Colby (*5*) in his Pigeon Roost report was the first to present a meaningful statement of the problem of nonuniqueness in the field.

If order can be applied to these data, as stated before, then the upper regime flows (plane bed and standing waves) should have resistance that is almost entirely due to grain resistance. Lower regime flows should have the same grain resistance plus an added form resistance. Streeter (*12*) relates the Darcy-Weisbach resistance coefficient, f $\left(= \frac{8gRS}{V^2} = 8\left(\frac{V_*}{V}\right)^2\right)$, to the Reynolds number, Re $\left(= \frac{4VR}{\nu}\right)$, through a relative roughness concept. Some values of resistance computed from this plot are given in table 1. These velocities computed on the basis of pipe curves in table 1 average about 15 percent too high.

TABLE 1.—*Values of resistance computed from Reynolds number-resistance plot forpipe flow for certain stations*

Station	$4VR/\nu$ x 10^{-6} at 1 ft.	$4R/k_s$ at 1 ft.	$V/V_* = C/\sqrt{g}$ Computed	$V/V_* = C/\sqrt{g}$ Measured	Difference Percent
Bernalillo, N. Mex.	1.23	3,300	22.3	20.1	+11
Byhalia, Miss.	1.12	2,600	22.0	18.9	+21
Logan, N. Mex.	1.16	6,200	23.4	24.4	− 4
Kaycee, Wyo.	1.26	2,000	21.6	16.1	+34
	At 2 ft.	At 2 ft.			
Bernalillo, N. Mex.	3.45	6,600	24.1	20.1	+20
Byhalia, Miss.	3.14	5,200	23.6	18.9	+25
Logan, N. Mex.	3.69	12,400	25.6	27.4	− 7
Kaycee, Wyo.	4.0	4,000	23.5	18.1	+30

One might suppose lower regime flows could be determined from the Einstein and Barbarossa bar-resistance plot. In figure 1, *D*, the rating computed from the bar-resistance plot for Bernalillo is compared with the measured discontinuous rating. The computed rating goes through the discontinuity as a smooth curve. Perhaps this is because the upper end of the

curve is based on data from the Elkhorn River near Waterloo, Nebr., which has both upper and lower regime flows (*2*). The bar resistance plot did not anticipate and therefore does not

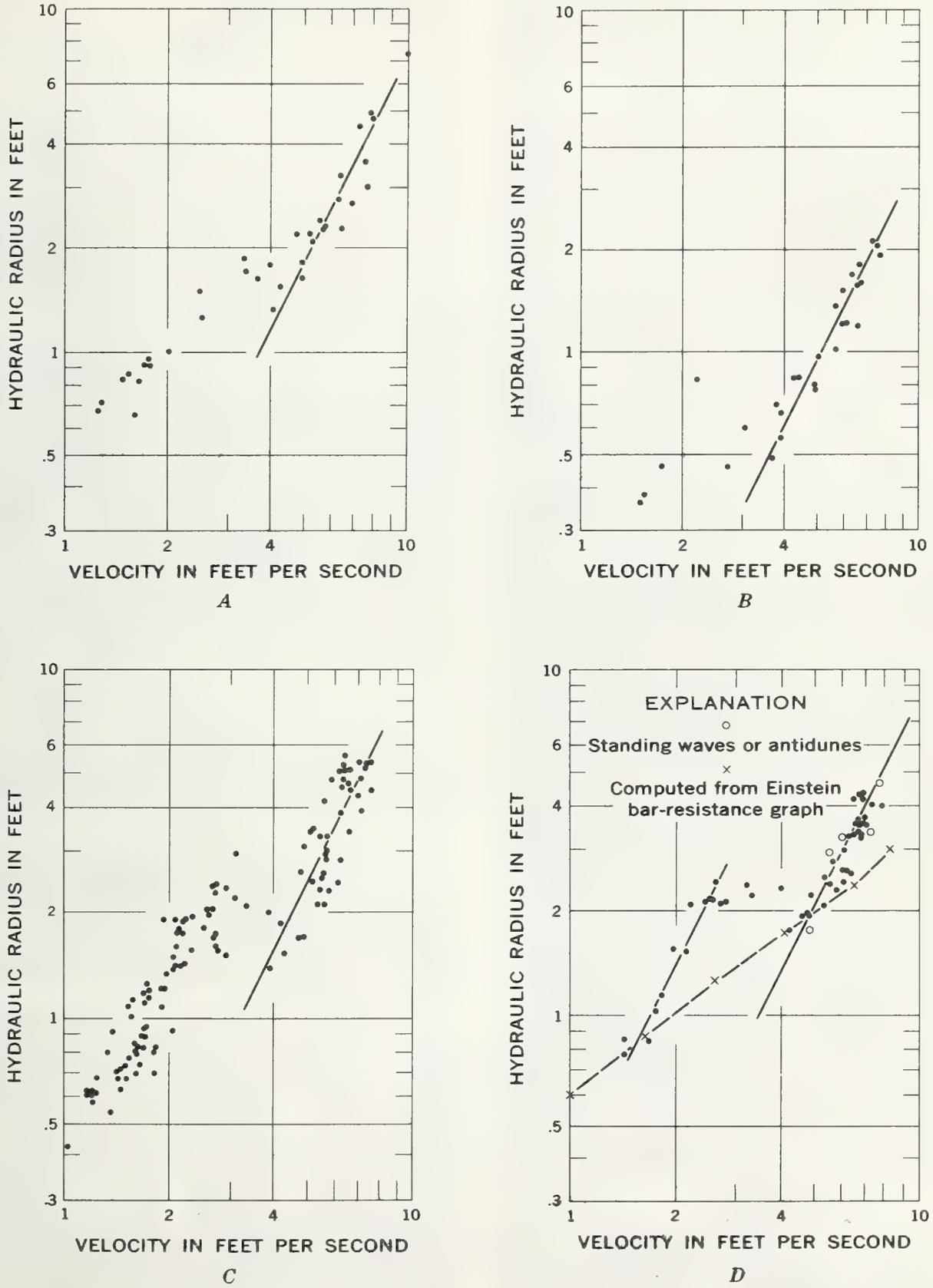

FIGURE 1. — Relation of velocity to hydraulic radius: *A*, Middle Loup River at St. Paul, Nebr.; *B*, Huerfano River near Undercliffe, Colo.; *C*, Pigeon Roost Creek near Byhalia, Miss.; and *D*, Rio Grande near Bernalillo, N. Mex.

handle the discontinuity. The bar resistance relation was developed for long reaches, and it is here applied to a section. A section plot for lower regime, perhaps including normal fixed boundary cobblestones and gravel reaches, might be sufficient. Resistance for upper regime flows seems possible through the pipe resistance type of approach.

The major problem still remains for the fluid mechanician to explain the mechanics and predict the occurrence of the discontinuity. The field man can fairly accurately tell regime of flow at the time of measurement by the appearance of the stream surface. The office engineer, somewhat less accurately, can use the appearance of a gage-height record for the same purpose. When these methods leave doubt, only intuition is left. As is shown by the plots, intuition can be wrong by an order of two. The intuitive method of guessing Manning's n is of no help, for the bed configuration after a flood event is of no help in determining regime at peak flow nor in determining if peak gage-height and peak discharge coincided.

For the gaging stations shown, the discontinuity is quite abrupt. There is a concomitant discontinuity in the shear-transport relation. Figure 2 shows a relation for the Rio Grande near Belen, N. Mex., Casa Colorado reach, of the Bagnold (*1*) shear against total sediment transport functions. The data are for measured suspended load only. By one interpretation (*6*), Bagnold predicts a rather abrupt eightfold increase of total load and explains it as development of a suspended load component. Bagnold's table 3 actually predicts a 3.54- to 3.9-fold increase for a 0.3 mm. sand, with a constant resistance coefficient. If $C/\sqrt{g}$ doubles in addition, the prediction becomes a 7.1- to 7.8-fold increase. Bagnold had flume data for total load only. I believe, but for a scale factor, Bagnold's flume results apply to the field case, and the two curves are for lower and upper regimes. The lower regime is with dunes on the bed. If, for dune flow, suspended load about equals bedload, Bagnold's curve seems to apply. If, in upper regime flows, bedload is negligible with respect to suspended load, as indicated by Simons (unpublished manuscript), then total load at Casa Colorada for both upper and lower regimes is about twice what Bagnold predicts. Interestingly, the slope of Bagnold's relation seems to be correct, because

$$\phi = K_1 \theta^{3/2} \doteq K_2 R^{3/2}$$

then if $$V = K_3 R^{1/2}$$

then $$\phi \doteq K_2 \left(\frac{V}{K_3} \right)^3 = K_4 V^3$$

which is Colby's (*4*) empirical formula relating total load to velocity. Colby's formula takes care of the discontinuity, for if velocity for a given depth doubles at the discontinuity, then load should increase by a factor of eight, as predicted by Bagnold and somewhat borne out by field data. However, velocity must be known in order to use the Colby formula, so that it begs the major question.

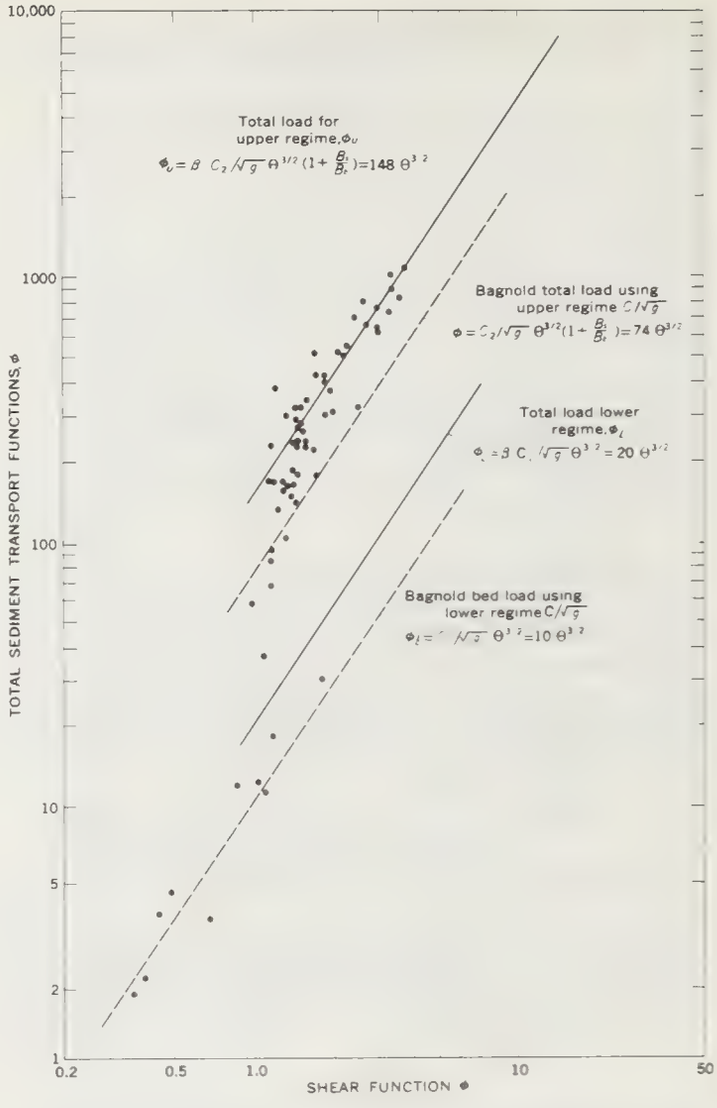

FIGURE 2. — Bagnold shear-transport functions: Rio Grande near Belen, N. Mex., Casa Colorado reach.

All discussion so far has concerned stations with little or no fine-material load. The addition of a charge of fine sediment has definite effects on the discontinuity in both the velocity and transport relations. Figure 3 shows the V-R relation for South Fork Powder River near Kaycee, Wyo. At this site large amounts of suspended fine material have been measured (*13*). Concentrations by weight of up to 120,000 p.p.m. of suspended material smaller than 0.062 mm. have been measured, and concentrations over 20,000 p.p.m. are common. A similar station is Canadian River near Logan, N. Mex. (fig. 4). On the other hand, Rio Grande at Bernalillo seldom has over 3,000 p.p.m. of sus-

pended fine material except during times of local tributary flow (*6*), Middle Loup at St. Paul seldom has over 3,000 p.p.m. of suspended fines at any time (*13*), and it is believed Huerfano River and Pigeon Roost Creek have similar amounts of suspended fines.

The effects of suspended fine sediment on bed form, and thus on the *V-R* relation, have been shown by flume studies (*11*). Apparently these effects combine to eliminate the abrupt discontinuity and produce a gradual change as evidenced by South Fork Powder and Canadian Rivers. The major effect of the fine material seems to be in changing the shape, and thus lowering the form resistance, of the dunes in the dune regime. The break occurs sooner, seems to be more gradual, and the data are decidedly less scattered, particularly for measurements in upper regime.

The slope of the average line for upper regime flows seems to be closer to two-thirds than to one-half for both the Kaycee and Logan stations. This would give them a constant Manning's n, and would indicate they more nearly approximate the pipe relation. Streeter (*12, p. 408*) states "a twofold change in (relative roughness) will cause a probable variation in f of less than 10 percent." This compares with a 12 percent change if Manning's n is constant.

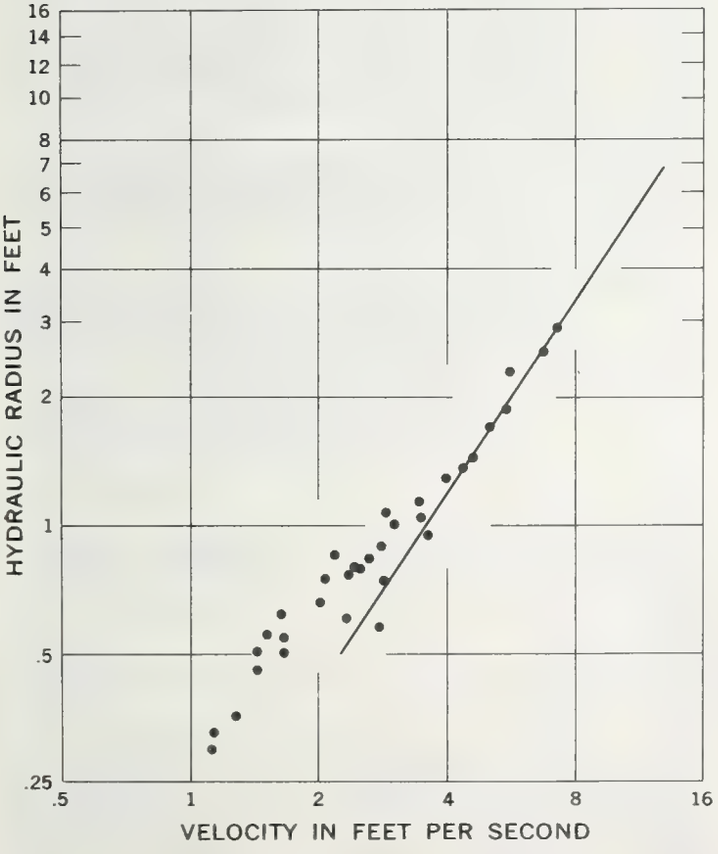

FIGURE 3. — Velocity-hydraulic radius (*V-R*) relation for South Fork Powder River near Kaycee, Wyo.

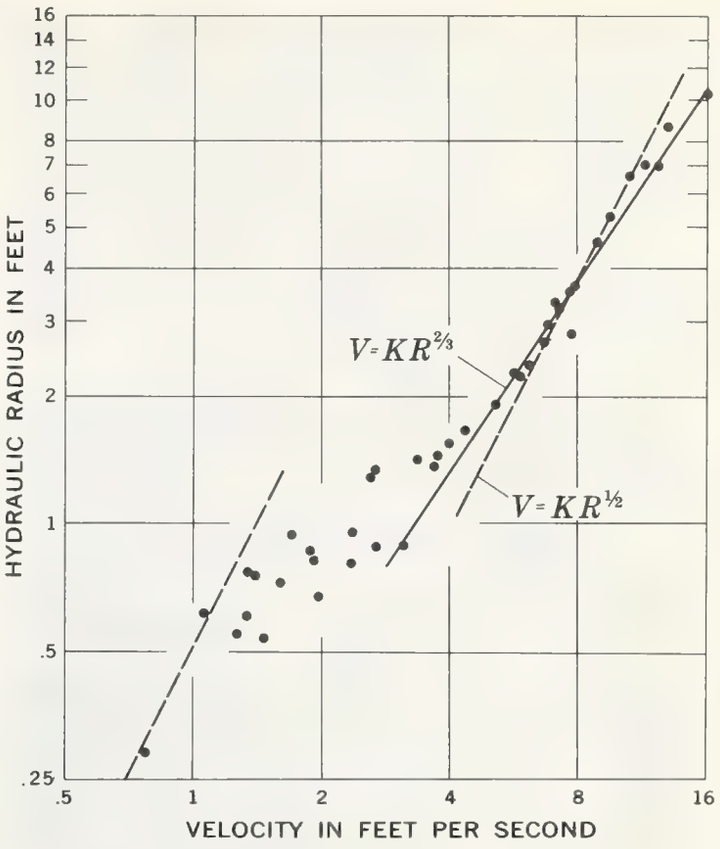

FIGURE 4. — Velocity-hydraulic radius (*V-R*) relation for Canadian River near Logan, N. Mex.

Therefore, probably neither Manning's n nor Chezy C is constant, but each is a fair approximation under certain conditions. Perhaps an alternative approach is through the use of the data to define a curve on a relative-roughness plot. This approach might result in not only a more rational discussion but also perhaps a better understanding of the definition of resistance to flow.

Conflicting evidence concerning the effect of suspended sediment on resistance to flow has been presented in the literature. Vanoni and Nomicos (*14*) conclude that sediment dampens the turbulence and thus reduces resistance to flow. Simons (personal communication) concludes that *fine* sediment increases antidune activity in upper regime flows and thus increases resistance. More or less valid arguments, depending upon the viewpoint of the individual, may be presented for and against either case. The available field data seem to indicate neither increase nor decrease of resistance for "in regime" flows. If resistance were a function of the amount of suspended sediment, then variation of suspended sediment should add one more unexplained variable to the *V-R* plot, thus increasing scatter about the mean line. Rather, the damping (or, perhaps better, suppression)

of turbulence seems to decrease scatter. Heuristically, one might argue the damping of turbulence tends to eliminate some instability phenomenon, which is characteristic of upper regime flows. Although resistance to flow seems to decrease with stage (perhaps because of a relative roughness factor), this does not support Vanoni's conclusion. Therefore, at this point one may believe almost anything concerning the effect of suspended sediment on resistance to flow and quote evidence in the literature to support his stand.

The addition of fine material load to the stream also has an effect on the shear-transport relation. Knapp (*9*) stated a condition for sediment that added rather than consumed energy. If this condition is met, "there is no limit to the concentration of such material that can be transported in suspension." This condition is similar to Bagnold's ratio of mean velocity of the sediment times slope to fall velocity (*1, p. 277*). Fine materials are those which meet Knapp's condition. There is not an unlimited supply of these in the bed. Therefore, they add energy to the stream, and this energy is expended in the transport of sand-size particles, which consume energy. The extra sand transport capacity with a fine material charge is borne out by both field and flume data.

In conclusion, in order to estimate velocity or transport in a sand channel stream, the regime of flow must be predicted. At this time it cannot. Further, it seems that in order to determine transport in the presence of a high concentration of fine material, one must measure it, a rather discouraging prospect.

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INFLUENCE OF SECONDARY FLOW ON LOCAL SCOUR AT OBSTRUCTIONS IN A CHANNEL

[Paper No. 36]

By Walter L. Moore, *professor and chairman*, and Frank D. Masch, *associate professor, Department of Civil Engineering, The University of Texas*

Introduction

It has long been recognized that an obstruction, such as a pier or bridge abutment, placed in an alluvial channel will disturb the flow and result in removal of some of the bed material in a localized region near the obstruction so that a scour hole is formed. In some instances the scour hole has developed to unexpected depths, causing undermining of piers and failure of bridge structures. In other instances piers have been extended to depths much greater than necessary to prevent undermining, which

has resulted in excessive construction expense. Frequently, the obstructions cause a contraction in the flow channel. Variations of the geometry of the contracted region due to local scour can influence the flow capacity of the contracted region.

A number of excellent laboratory investigations have been made of these local scour problems (*3,4,5,6;7,10*). These have provided information on the shape of the scour pattern, on methods for predicting the depth of local scour, and in some instances on methods for controlling the depth of scour by selecting favorable shape parameters or by installing special scour control devices.

In these studies some limited observations were made of the details of the flow pattern at the obstruction. The spiral vortex forming in the scour hole at a circular pier is well known. In this paper an attempt will be made to develop a better understanding of some of the characteristics of the three-dimensional flow patterns occurring at such obstructions, which may lead to a better basis for devising methods to control the local scour.

Secondary Flows

For the purposes of this paper a secondary flow will be considered as one for which the velocities differ markedly from those indicated by the simple two-dimensional methods of analysis commonly applied to determine the flow around an object. Figure 1 shows the results of the simple analysis for flow past a cylindrical pier with a uniform approach velocity. If the axis of the pier is placed vertical, the velocities will all be in horizontal planes and their magnitudes given by potential flow theory except in the wake zone and in the boundary layer. Along the upstream element of the cylinder, which is the stagnation line, the velocity is zero and the pressure is a maximum equal to the dynamic pressure $\rho U_0^2/2$ for the approach velocity U_0.

Moving from the stagnation line around the cylinder, the tangential component of velocity increases to a maximum of about twice the free-stream velocity at the maximum breadth point. In this vicinity the flow separates from the cylinder, producing the wake zone of relatively low velocities. The normal pressure on the surface of the cylinder decreases from its maximum at the stagnation line to zero, or the static pressure, at $\theta=30°$ for an inviscid fluid and a somewhat greater angle for a real fluid. The normal pressure then continues to decrease and becomes negative, that is, less than the free-stream pressure by an increasing magnitude, until the separation point is reached. Within the wake zone the negative pressure may be considered as essentially constant and of the same magnitude as at the separation point. For these conditions the tangential velocity U_t will be given by

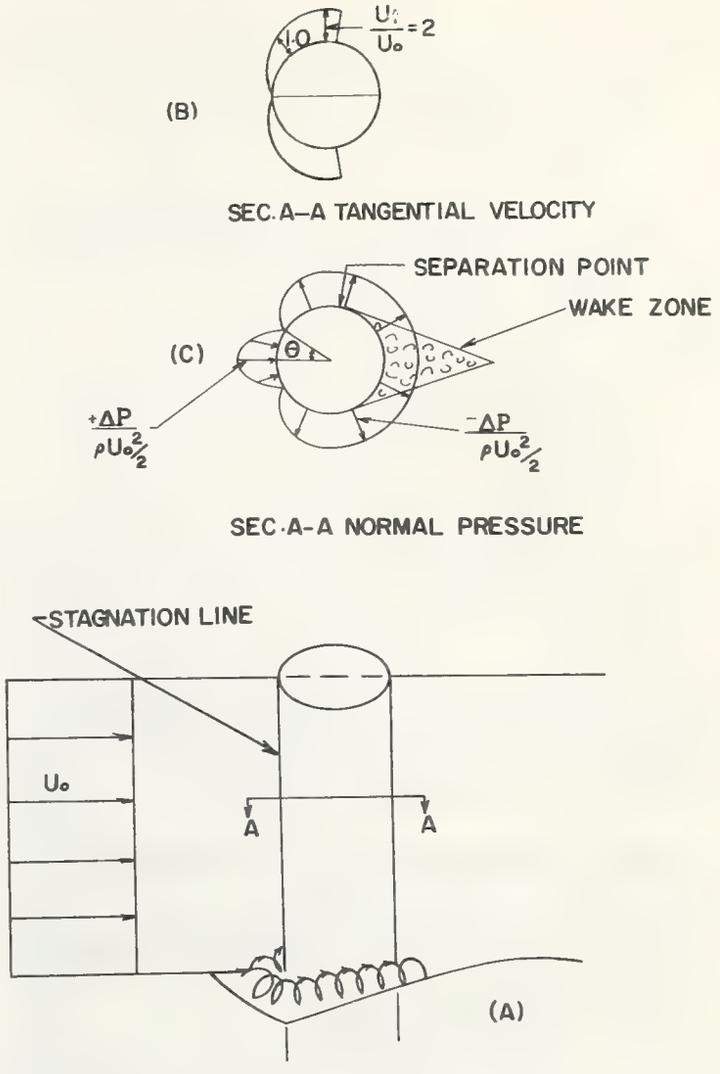


FIGURE 1.—Cylindrical pier with uniform approach velocity.

$$\frac{U_t}{U_0}=2\sin\theta;\left(0<\theta<\frac{\pi}{2}\right) \tag{1}$$

where U_0 is the free-stream velocity and θ the angle measured from the direction of the approaching flow. The variation in piezometric head around the surface of the cylinder is given by

$$\Delta h=h-h_0=\frac{U_0^2}{2g}(1-4\sin^2\theta);\left(0<\theta<\frac{\pi}{2}\right) \tag{2}$$

where the piezometric head h is the sum of the pressure head and elevation head, or

$$h=\frac{p}{\gamma}+y \tag{3}$$

Now consider the case where the approach velocity is not constant along the length of the cylinder but is larger at the upper end, as shown in figure 2,*A*. Since U_0 is now a function of y, it is clear that along the stagnation line the pie-

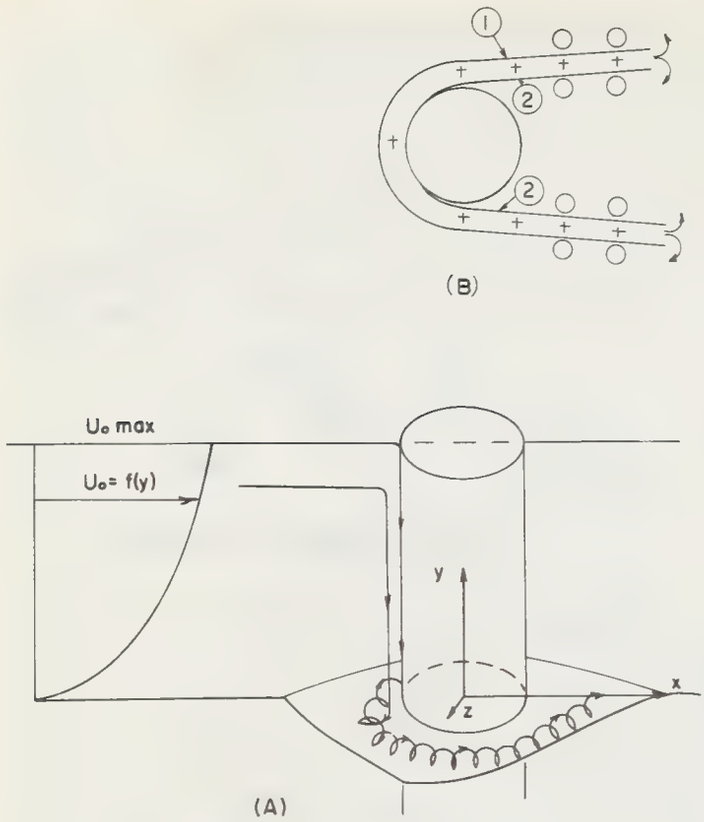


FIGURE 2. — Cylindrical pier with nonuniform approach velocity.

zometric head is also a function of y. If equation 2 is differentiated with respect to y and $\theta=0$ is substituted, it gives

$$\frac{\partial h}{\partial y}=\frac{\partial}{\partial y}\left(\frac{U_0^2}{2g}\right);\ (\theta=0) \tag{4}$$

This gradient of piezometric head in the y direction implies an acceleration of the fluid along the stagnation line in the negative y direction. If, for simplicity, we neglect viscous effects and consider a stream line running downward along the stagnation line of the cylinder, Euler's equation states that

$$\frac{\partial h}{\partial y}=-\frac{\partial}{\partial y}\left(\frac{V_y^2}{2g}\right) \tag{5}$$

which with equation 4 yields

$$\frac{\partial}{\partial y}\left(\frac{U_0^2}{2g}\right)=-\frac{\partial}{\partial y}\left(\frac{V_y^2}{2g}\right) \tag{6}$$

This may be integrated along the stagnation line, which is a secondary flow stream line, to yield

$$\frac{V_y^2}{2g}=\frac{U_0^2{}_{\max}}{2g}-\frac{U_0^2}{2g} \tag{7}$$

the constant of integration having been determined by setting $V_y=0$ where $U_o=U_{o\,\max}$, which occurs at the top of the cylinder. This gives the interesting result that at the bottom of the cylinder where U_o is zero, the downward velocity along the stagnation line V_y is equal to the surface velocity of the stream, $U_{o\,\max}$, or $V_y=U_{o\,\max}$ at $y=0$.

These concepts give the essential basis for a mechanism producing a strong downward vertical jet along the upstream nose of the pier and account for the fact that the maximum depth of the scour hole occurs at the most upstream point of the pier rather than at the maximum breadth points, as would be indicated by the velocity pattern for two-dimensional flow. As shown in figure 2,*A*, this downward secondary flow contributes to the formation and the sustaining of the spiral vortex in the scour hole around the upstream side of the pier.

What would happen if the cylinder rested on a rigid bed? In this case the downward secondary flow would not reach a maximum at the base of the cylinder but would encounter a second stagnation point at the bed, resulting in an increase in pressure and causing the central stream line to turn upstream. It would continue upstream until the resistance of the oncoming flow reduced the velocity to zero, where the streamline would split and move around the cylinder. This upstream flow at the base of cylinders in a boundary layer has been observed by dye traces (*12*) and also in studies of cylindrical total-pressure probes in flows with a transverse gradient of total pressure (*11*). The stream line pattern on the central plane is shown in figure 3 for the streamline with maximum approach velocity and for one streamline starting at a slightly lower level. It is clear that the actual pattern will be a function of the velocity distribution in the approaching flow. The pattern presented in figure 3 is not intended to

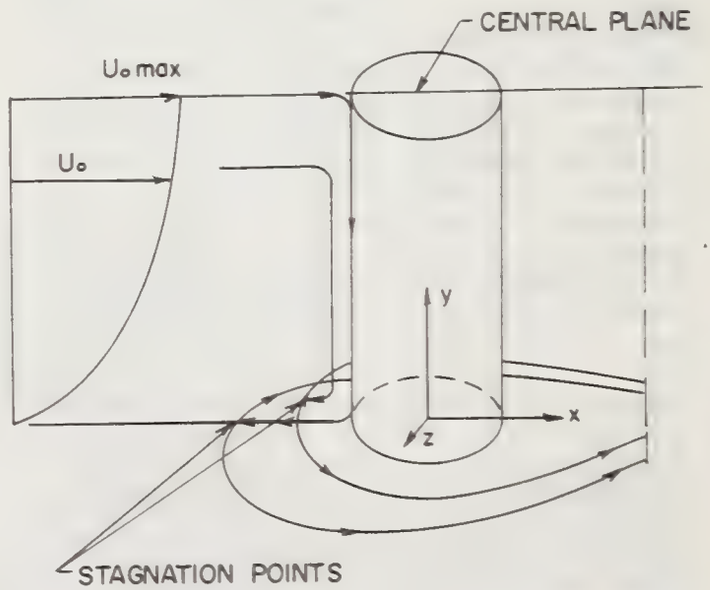


FIGURE 3. — Cylindrical pier with nonuniform approach velocity — rigid bed.

imply that water moving along the surface of the stream approaching a bridge pier will actually plunge downward along the stagnation line all the way to the bottom. Due to unsteadiness present in the flow, this ideal picture would not be realized and any particular fluid particle starting on the central plane would undoubtedly slide around the pier before being deflected a great distance in the downward direction. Baines (*1*) has described a similar flow on the upwind side of models of tall buildings tested with a velocity gradient in the approaching flow.

As seen in figure 1,*C*, the pressure for the entire forward part of the cylinder is increased, causing the piezometric head in this region to be larger than the static head. The magnitude of the piezometric head decreases in the downward direction because of the velocity gradient in the primary flow.

Behind the static pressure point, where the pressure is reduced and the piezometric head is less than in the free stream, the negative pressures are greater near the top, causing a gradient of piezometric head in the reverse direction that induces an upward flow in the wake zone. The entire pattern may be visualized as shown in figure 2,*B*, where line 1 represents a sheet of vorticity wrapped around the leading edge of the cylinder with downward flows, as indicated by crosses on the inside of the line, and upward flows as indicated by circles on the outside of the line. Two other sheets of vorticity, indicated by lines 2, attached near the static pressure elements of the cylinder would have opposite sense with downward flows on the outside of the lines and upward flows on the inside. Upward flow in the separation zone has been observed by Masch (*13*).

This problem of secondary flows has been analyzed in detail by Lighthill (*8,9*) for the simplified case of the flow of inviscid fluid with a constant velocity gradient past an infinitely long cylinder with no separation. A refinement of this analysis by Toomre (*14*) takes into account the viscous effect of the boundary on the secondary flow, which gives the secondary flow field in the plane of symmetry in front of the cylinder as shown in figure 4. In this figure, R is the Reynolds number, V_y the secondary flow velocity parallel to the cylinder, a the radius of the cylinder, and A the velocity gradient in the expression

$$U_o = U_1 + Ay$$

The velocity U_1 is a reference velocity at some arbitrary level.

As an example of the result of this analysis, consider a stream 10 feet deep with a surface velocity of 8 feet per second and a velocity near the bottom of 3 feet per second that corresponds

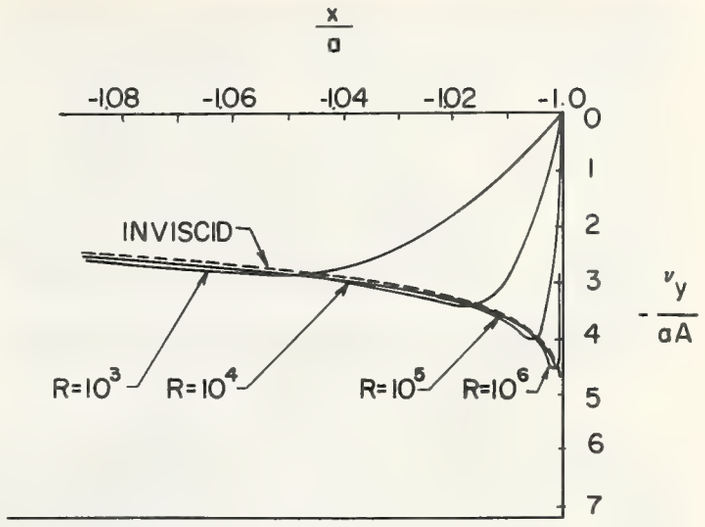


FIGURE 4.— Profiles of viscous secondary flow in the plane of symmetry in front of circular cylinder.

to A= 0.5. From figure 4, $V_y/aA = -4$, yielding $V_y = 6$ feet per second downward velocity near the upstream element of a 6-foot diameter cylinder. This simplified analysis for an infinitely long cylinder thus gives results with a reasonable order of magnitude.

Hawthorne (*2*) presents mathematical solutions for the secondary flow at sharp-nosed struts and airfoils in a velocity gradient flow. He also describes limited scour experiments in a sand bed, which support his analysis.

For a bridge abutment the secondary flow pattern may be visualized as sketched in figure 5. Line 1 again represents a sheet of vorticity

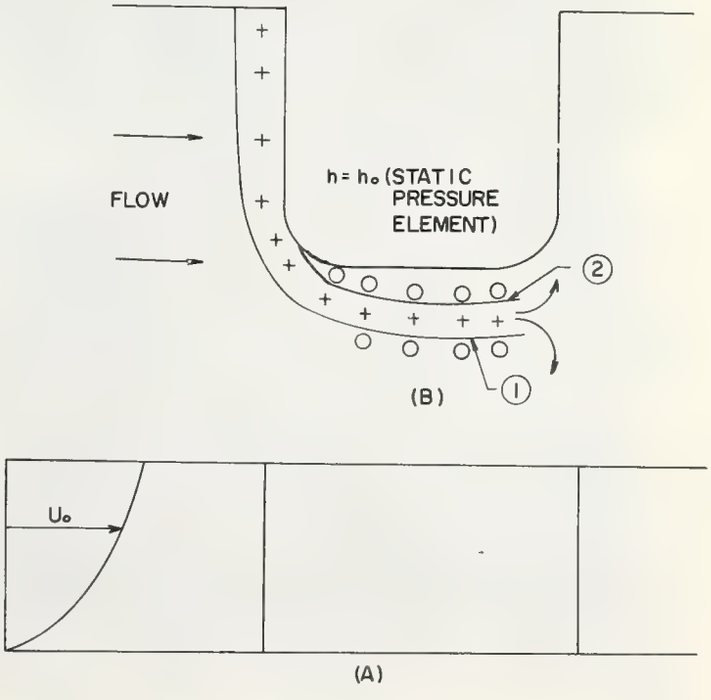


FIGURE 5. — Secondary flow at a bridge abutment.

wrapped around the forward part of the abutment with downward velocities, represented by crosses inside the line. This line of vorticity would curve upward and end at the water surface near the bank. Line 2 represents a sheet of vorticity of opposite sense attached to the abutment near the static pressure element with downward velocities, represented by crosses, outside the line and upward velocities represented by circles inside the line.

Relation Between Secondary Flows and Observed Scour Patterns

The implications of these secondary flows on local scour patterns at obstructions are examined, and the flows compared with the scour patterns observed in the laboratory. From earlier discussion, it is apparent that the strongest secondary flow is downward along the most forward element of the cylinder. This corresponds to the point of maximum observed scour depth in laboratory studies (fig. 6). Further, it will be noted that the two trailing sheets of vorticity as shown in figure 2,B, imply strong downward secondary flows in the regions corresponding to the scour tails extending downstream on each side of the scour hole. The vorticity represented by line 2 implies inward flows along the bottom, accounting for the piling up of sediment downstream and inside of the two scour tails as has been observed in the laboratory.

In the case of the abutment, the secondary flows indicated by figure 5,*B*, imply that the deepest scour would occur between the two sheets of vorticity and that material would be moved toward the abutment by the vortex system represented by line 2. This pattern has also been observed in the laboratory as shown in figure 7.

Proposals for Control of Scour

The concepts of secondary flows as the cause of scour suggest several possible methods for limiting the scour depth at piers. Four basically different ideas will be discussed briefly, each one of which raises a host of questions that must be answered before a suitable design could be chosen.

Horizontal Fins

Horizontal fins placed below the streambed and extending outward from the pier have been proposed and investigated as a means of limiting scour hole development. Based on the concepts of this paper, it appears that the lateral extent of these fins might be reduced by putting a lip at the outer edge to deflect the secondary flows back toward the surface and away from the bottom. It would also seem that a fin or a shoulder placed above the streambed would be helpful in breaking up the downward secondary flows and thus reducing the scour. However, with the fin placed above the bed, the secondary flows may develop again below the fin due to the high velocity gradient present near the streambed.

A Pier Located Upstream from the Main Pier

An auxiliary pier placed upstream has been proposed before, mainly with the idea of creating a scour hole upstream and far enough away to prevent damage to the main structure. Based on the concepts of this paper, it follows that such an upstream pier could be quite small, thus reducing its own scour hole. It could be rather short so the top would be submerged; the only requirement being that the pier be long enough to destroy the velocity gradient in the flow approaching the nose of the main pier. It is believed that the secondary flow concepts will be helpful in understanding and predicting the action of such a scour control device.

A Sharp Leading Edge

Because a blunt-nosed pier gives a larger region of retarded flow, it also results in a larger and stronger secondary downward flow at the nose, thus increasing the tendency to scour. A sharp nose with gradually tapering sides will produce a less intense secondary flow and reduce the maximum depth and extent of scour.

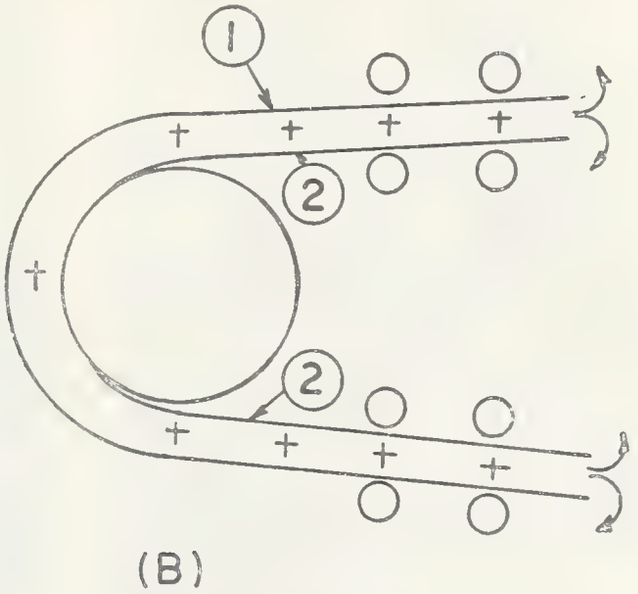
A Sloped Leading Edge

It is known that if a cylinder is placed in a flow with its axis swept back in the direction of flow, a flow will be induced along the leading edge in the direction of the sweep-back. This raises the intriguing possibility that a pier swept backward and upward at the proper angle could produce a secondary flow just equal and opposite to that induced by the velocity gradient in the approaching flow. It would appear that this swept-back leading edge would be needed only near the streambed where it could stop the downward flow along the leading edge before it attacked the bed of the stream. The effectiveness of this swept-back leading edge would probably depend upon its angle, its location relative to the streambed, the sharpness of its leading edge, and on how far it extended below the streambed.

Discussion

The concepts of secondary flow as developed here give some possibilities for the development of effective methods of scour protection for piers. Research is currently under way at the University of Texas to investigate in detail the nature of the secondary flows that develop

around obstructions in velocity gradient fields where the bed is rigid. Plans are being made for a systematic study of the effects of these secondary flows on movable beds and of devices to control the depth of scour by controlling the secondary flows.



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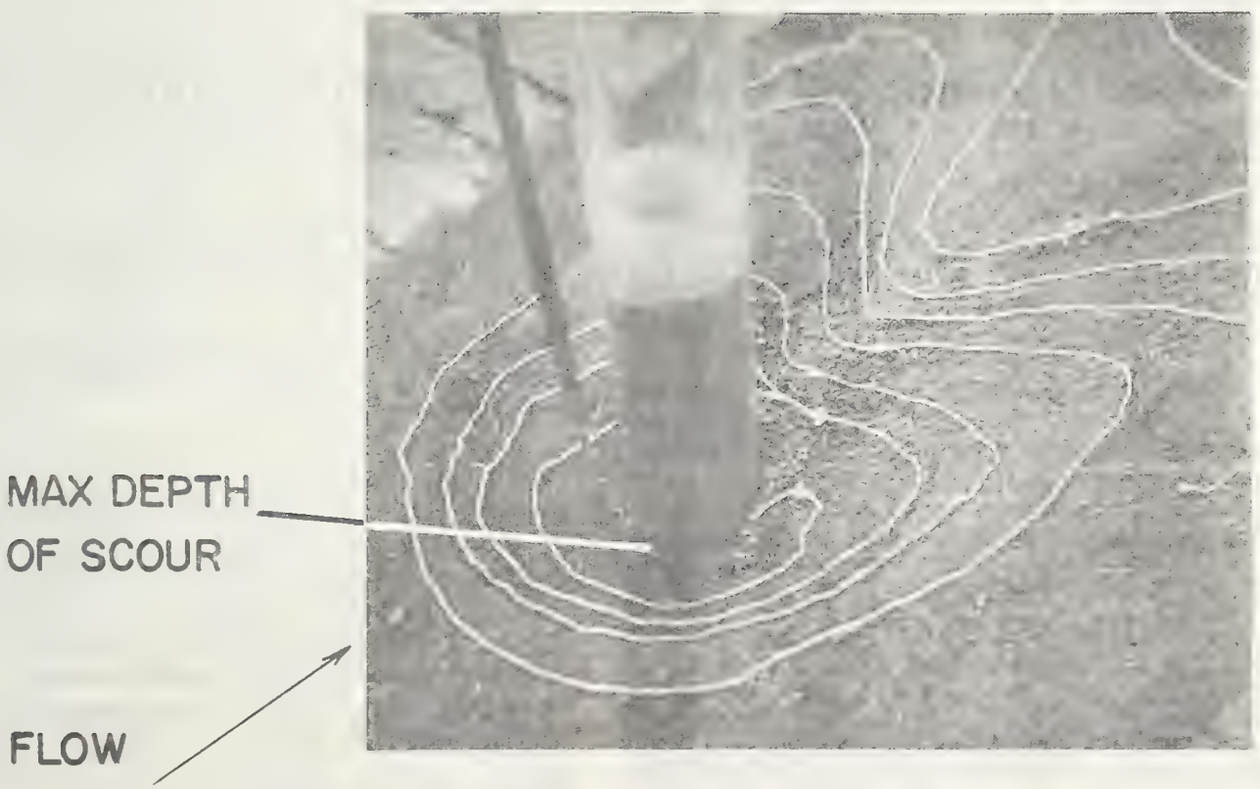

FIGURE 6. — Observed scour pattern at a round-nosed pier.

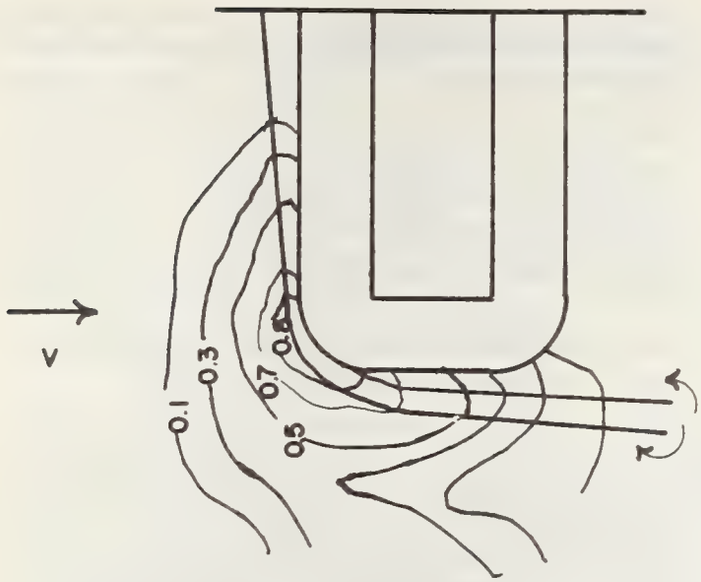

FIGURE 7. — Observed scour at a bridge abutment.

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LABORATORY STUDY OF AN ALLUVIAL STREAM AT CONSTANT DISCHARGE[1]

[Paper No. 37]

By JOHN F. KENNEDY, *assistant professor of hydraulics, Hydrodynamics Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge;* and NORMAN H. BROOKS, *professor of civil engineering, California Institute of Technology, Pasadena,* and during 1962-63, *visiting professor, Hydrodynamics Laboratory, Massachusetts Institute of Technology*[2]

Abstract

A laboratory study in a 60-foot flume was made to determine the hydraulic and sediment transport characters of an alluvial stream when the discharge is constant and the depth varies. These experiments supplement those of Vanoni and Brooks in which the discharge was varied with the depth constant.

For the given unit discharge (0.50 c.f.s./ft.) and bed sand (mean diameter 0.14 mm.) it was found that two different velocities and sediment transport rates are possible for certain values of slope or bed shear stress; however, this multiplicity occurs only in the range of slope or shear stress where major changes in bed configuration and roughness occur.

Stream morphology is discussed in light of these and other laboratory results, with emphasis on which variables are independent and which are dependent.

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Background and Objectives

In 1955 Brooks (*2*) reported a laboratory study of the mechanics of open-channel flow over a sand bed. The results of this investigation led him to question some widely held concepts about the relation between the sediment transport rate, depth and velocity of flow, and slope of alluvial channels. Brooks' conclusions, as restated by Vanoni and Brooks (*19*), are as follows:

1. In the laboratory flumes it was found that neither the velocity nor the sediment transport rate could be expressed as a single-valued function of the bed shear stress, or any combination of depth and slope, or bed hydraulic radius and slope. This contradicts assumptions which have commonly been held for some years to the effect that knowledge of the slope, channel geometry, and bed material of a stream were sufficient to predetermine its flow and sediment-transporting characteristics.
2. The cause of the non-uniqueness cited in conclusion no. 1, above, is the extreme variation in channel roughness caused by the variable nature of the bed configuration. In general, the flows at low velocities were accompanied by high, rugged dunes, while those at high velocity were associated with flat beds, with sand wave phenomena occurring at intermediate velocities.
3. On the basis of experiments to date, it appears that the depth and velocity may logically be used as independent variables, along with the grain size of the bed material. Knowing these quantities, it is found for the flumes that the slope, bed shear, friction factor, and sediment discharge are all uniquely determined. It was also found possible to consider the water and sediment discharges as independent variables, and from these to determine the depth, velocity, friction factor, and slope.
4. An examination of the results of these experiments yielded the following qualitative relationships:

a. For constant discharge, q, an increase in the sediment discharge, q_s, requires a decrease in the depth, d.

b. For a given slope and discharge, two depths of flow are possible. When q_s is low, the bed is covered with dunes, d is large, and the velocity, U, is small. When q_s is high, the bed is flat, d is small, and U is large.

c. If q is to be increased without changing q_s, then an increase in d is necessary, although this increase is relatively less than in q.

d. For a given q, the largest values of the bed friction factor, f_b, are associated with the lowest values of q_s.

e. When U is increased with d constant, f_b generally decreases, the slope S and the bed shear velocity U_{*b} may either increase or decrease, and the sediment discharge concentration, C, increases until the sand wave stage is reached.

f. When d is increased with U constant, f_b and C both decrease.

g. [3] The bed shear velocity, U_{*b}, changes less than any other quantity. It may be expected, therefore, that U_{*b} is not a good variable from which to determine the flow and sediment transport characteristics.

The discussion of Brooks' paper (*2*) was, for the most part, vigorously skeptical. In his closing discussion (*2*), Brooks presented the results of a new, larger group of experiments [also reported by Vanoni and Brooks (*19*)], some of which were designed to answer specific objections raised by the discussers. The new investigation covered a wider range of velocity at two different constant depths and used three sands with wider size distributions than the two fairly uniform sands he originally used. The results of the new experiments were in complete agreement with Brooks' original conclusions, which were pertinent. Those regarding the variation of sediment transport rate, slope, and friction factor with velocity for a constant discharge were neither proved nor disproved, because of the manner in which the runs were scheduled: the depth was fixed for each series of runs and the discharge was varied.

The objective of the experiments reported here was to verify Brooks' conclusions for the case of constant discharge. The main point to be resolved was whether two (or possibly three) different depths of flow are possible for a given discharge and slope. This question was the subject of considerable controversy in the discussion of Brooks' paper and was not settled by the later experiments (*2, p. 582* and *p. 594*). In the present inquiry, eight runs were conducted at a constant unit discharge of 0.50 c.f.s. per foot. For a more detailed description of these experiments the reader is referred to a report by Kennedy (*10*).

At the end of the paper stream morphology is discussed in the light of these experimental flume results.

Apparatus and Procedure

Only a summary description of the apparatus and procedure will be presented here. Complete details and thorough discussions of the apparatus, the techniques used in measuring the various quantities, and the procedure used in reducing the data have been given by Brooks (*2*), and by Vanoni and Brooks (*19*).

The experiments were performed in a 33.5-inch wide, 60-foot long flume. The flow circuit of the flume is closed, so the water and sediment are continuously recirculated and the mean depth of flow in the open channel is controlled by the amount of water in the system. The entire flume is mounted on a truss that is pivoted near one end and supported by a jack at the other end so the flume slope can be set at any desired value and easily adjusted during the course of a run. A point gage mounted on a carriage that moves along rails on the flume was used to measure the elevations of the water surface and sand bed relative to the flume. Water surface elevations were measured during the run at 4-foot intervals along the flume, and bed elevations were measured after the pump had been stopped and the bed leveled in 4-foot reaches with a special leveler. The discharge

[3] Conclusion 4g is based primarily on the experiments by Vanoni and Brooks (*19*).

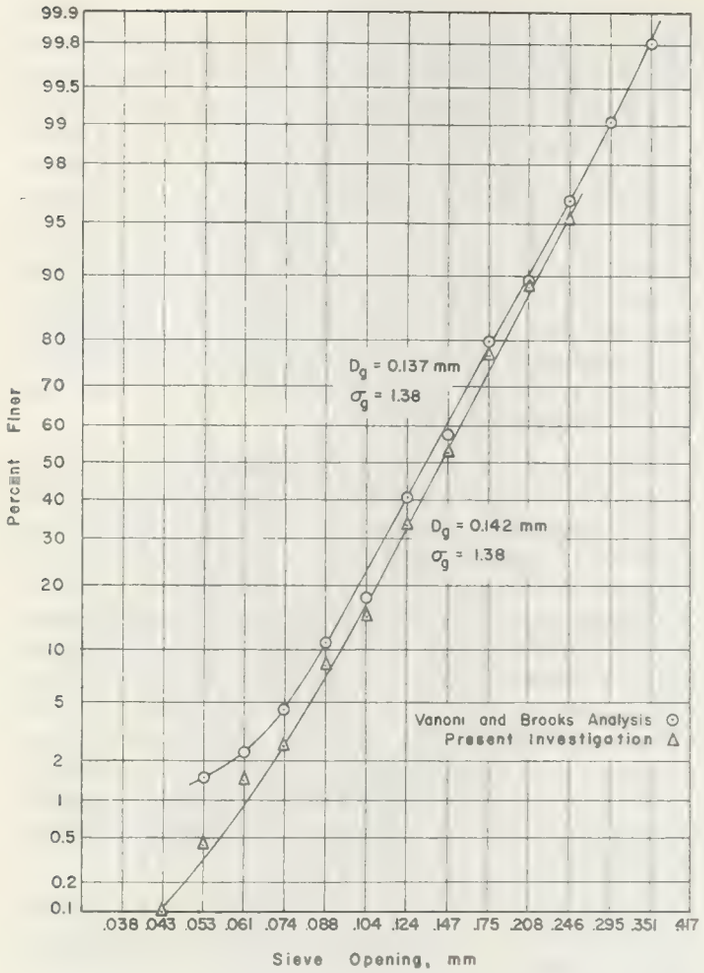

FIGURE 1. — Sieve analyses of the bed material.

was measured with the venturi meter located in the return pipe. To determine the total sediment transport rate, 12 or more samples of 1 liter each were siphoned through special samplers from the vertical section of pipe above the pump in the flow circuit. The individual 1-liter samples were then passed through filter paper, and the sediment retained was dried and weighed. A sieve analysis was performed on the composite of all samples for each run.

Sand Characteristics

The sieve analysis of the bed sand, plotted on logarithmic-probability paper in figure 1, shows that the geometric mean sieve diameter, D_g, was 0.142 mm., and the geometric standard deviation, σ_g, was 1.38. This is compared in figure 1 with the sieve analysis of the same sand when it was being used previously in the same flume by Vanoni and Brooks (*19*). There was a slight increase in D_g, no change in σ_g, and a significant decrease in the amount of fine material. For example, at the conclusion of the present experiments there was only about 76 percent as much material finer than the 170-mesh (0.088 mm.) sieve as during the experiments by Vanoni and Brooks, and for the smaller sieve sizes the losses were even greater. Most of the loss probably occurred when the flume was periodically drained to be refilled with clean water. This coarsening of the bed material will be cited later to explain a discrepancy in the transport rate between the results of the present experiments and those of Vanoni and Brooks.

Experimental Results

The principal measured and computed quantities from the laboratory experiments are summarized in tables 1 and 2. Quantities with subscript b refer to the bed section, as computed from the sidewall correction procedure, described by Vanoni and Brooks (*19*). Table 1 is self-explanatory except for the quantity, f_b/f_b', the friction factor ratio. In this quantity, f_b' is the friction factor predicted by the Moody pipe-friction diagram, using the geometric mean sieve diameter of the bed material, D_g, as the equivalent sand roughness, and $4r_b$, where r_b is the bed hydraulic radius, in place of the pipe diameter for the characteristics length. Thus, f_b' is the friction factor predicted by the Moody diagram for flow of clear water over an immobile flat sand bed. The friction factor ratio, f_b/f_b', is a measure of the effect of dunes or other bed features on the friction factor of the flow and serves as a partial quantitative description of the bed configuration; it should be approximately unity for flow over a flat bed. The use of this friction factor ratio in analyzing the roughness of alluvial channels was first suggested by Vanoni and Brooks (*19*). Detailed discussions of the friction factor ratio and its significance have been presented by Kennedy (*11*) and Taylor and Brooks (*18*). Values of f_b/f_b' obtained here are consistent with previous investigations (*18*).

The various types of bed configuration listed in column 18 of table 1 have been discussed by Brooks (*2*) and by Vanoni and Brooks (*19*). A dune-bed configuration, illustrated in figure 2, occurred for all runs of this investigation with velocity less than 1.5 f.p.s. The average wave lengths and heights of dunes of these runs are listed in table 2. The lengths were obtained by counting the number of dune crests along a measured line segment parallel to the flume. Dune heights were computed from measured elevations of troughs and crests of representative dunes.

Discussion

Variation of Friction Factor and Bed Shear Stress

The variations of slope S, bed shear velocity U_{*b}, and bed friction factor f_b, with changing mean velocity U, are shown in figure 3. These relations were obtained by first fitting a curve to the f_b data, and then computing values of S

TABLE 1.—*Summary of hydraulic and sediment transport data from laboratory experiments*

Sand characteristic: $D_0 = 0.142$ mm, $\sigma_g = 1.38$

Run No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Run No.
	Depth d	Mean Velocity U	Unit discharge q	Froude number F	Hydraulic radius r	Slope S	Shear velocity U_*	Friction factor f	Bed hydr. radius r_b	Bed shear velocity U_{*b}	Bed friction factor f_b	Friction factor ratio $\frac{f_b}{f_b}$	Water temp. T	Discharge concentration C	Sed. disch. rate q_s	Analysis of sediment load: D_0	Analysis of sediment load: σ_0	Bed configuration	
	Ft.	*Ft./sec.*	*C.f.s./ft.*		*Ft.*		*Ft./sec.*		*Ft.*	*Ft./sec.*			°C.	*G./l.*	*Lb./min. =ft.*	*Mm.*			
3–1	0.550	0.91	0.500	0.22	0.395	0.00056	0.084	0.068	0.503	0.095	0.086	4.85	19.5	0.014	0.026	0.103	1.60	Dunes	3–1
3–4	.441	1.14	.503	.30	.335	.00145	.125	.096	.414	.139	.119	6.60	18.4	.39	.73	.100	1.46	Dunes	3–4
3–2	.373	1.35	.504	.39	.294	.00206	.140	.086	.352	.153	.102	5.67	18.4	1.42	2.68	.096	1.43	Dunes	3–2
3–7	.345	1.45	.500	.44	.277	.00198	.133	.067	.324	.144	.079	4.34	25.1	1.13	2.12	.097	1.47	Dunes	3–7
3–5D [1]	.340	1.47	.500	.44	.273	.0016	.119	.052	.315	.126	.060	3.30	25.3	.98	1.84	.085	1.45	Dunes (S.W.) [2]	[1]3–5D [1]
3–5F [3]	.254	1.97	.500	.69	.215	.0025	.131	.036	.234	.137	.039	2.05	25.3	1.71	3.20	.102	1.49	Flat (S.W.) [2]	[3]3–5F [3]
3–6a	.235	2.13	.501	.78	.201	.00198	.113	.023	.209	.115	.023	1.22	25.7	1.57	2.95	.127	1.41	Flat	3–6a
3–6	.233	2.15	.501	.78	.200	.00207	.115	.023	.209	.118	.024	1.25	25.2	1.41	2.65	.128	1.38	Flat	3–6
3–6b	.228	2.21	.504	.82	.196	.00221	.118	.023	.205	.121	.024	1.24	25.4	1.74	3.29	.....	.....	Flat	3–6b

[1] Dune section of flow with a long sand wave.
[2] S.W. = Sand wave developed on bed.
[3] Flat section of flow with a long sand wave.

TABLE 2.—*Summary of data from dune measurements*

Run No.	Average length	Average height
	Ft.	*Ft.*
3–1	0.43	0.049
3–4	.46	.064
3–2	.48	.070
3–7	.42	.066
3–5D	.50	.061

and U_{*b} corresponding to points on the f_b curve; thus, the three curves are consistent with each other. Between the points for the run with a sand wave, the curves are shown as dashed lines because there is some question of whether or not uniform equilibrium flows over a sand of this size can exist at these intermediate velocities. For instance, for run 3–5, the amount of water was adjusted to yield a uniform depth of 0.30 ft. with mean velocity of 1.67 f.p.s. over the full length, but instead the sand wave phenomenon produced a depth of 0.34 ft. over the dune section and 0.25 ft. over the flat-bed section.

In figure 3 it is seen that the bed friction factor increases from 0.086 to 0.119 as the velocity increases from 0.91 to 1.14 f.p.s. With further increase in velocity, the bed friction factor decreases to 0.023 at a velocity of about 2.1 f.p.s. An initial increase of bed friction factor with velocity for this sand has also been observed by Vanoni and Brooks (*19*) for runs at a nearly constant depth of about 0.54 ft., but not for runs at a depth of about 0.24 ft. Thus, it appears that for some depths there is a velocity for which the roughness of the dune configuration is a maximum.

An important point in these experiments is the fact that the bed friction factor varied by a factor greater than 5 from 0.119 down to 0.023. The variation in f_b was due primarily to changes in the roughness of the bed as its configuration changed. As the velocity was increased above a certain value (about 1.2 f.p.s. for these experiments), the shape and arrangement of the dunes changed in such a way that the roughness of the bed decreased. When the bed became flat, the only roughness was that of the individual sand grains and was near the irreducible minimum. However, a small further decrease in the friction factor could have occurred at higher velocities with larger sediment concentrations. The suspended sediment apparently modifies the turbulence structure of the flow and thereby reduces the friction factor. This effect has been described by Vanoni and Brooks (*19*) and by Vanoni and Nomicos (*21*).

The large variation in the bed friction factor has an important effect on the relation between U and S, and between U and U_{*b}. As shown in figure 3, for a constant discharge the slope in-

FIGURE 2. — Dune configuration of run 3-2. Flow direction left to right. Full 33.5-inch width of the flume is shown. Bed friction factor, f_b=0.102.

creases with velocity throughout most of the dune regime, then decreases at the upper end of the dune regime. In the flat bed regime, the slope again increases with velocity. Consequently, an identical value of slope, S=0.00198, was observed for two different runs: Run 3–7 (U=1.45 f.p.s., q_s=2.12 lb./min.-ft., dune bed) and run 3–6a (U=2.13 f.p.s., q_s=2.95 lb./min.-ft., flat bed). These two runs confirm, for this sand and this unit discharge, Brooks' conclusion No. 4b that for a given discharge and slope, two different velocities are possible. This type of nonuniqueness is illustrated schematically in figure 4.

Brooks' conclusion No. 4b regarding the multiplicity of velocity as a function of slope for constant discharge is valid only for a certain range of velocity and a corresponding range of slope. Since this multiplicity is a result of major changes in the bed configuration, two or more values of velocity for a given slope can occur only in the range of U and associated range of S from slightly less than that at which the transition from dunes to sand wave occurs to slightly greater than the velocity at which the bed configuration changes from sand wave to flat. For the constant discharge experiments reported here, this range of slope is S=0.0018 to 0.0021. The multiplicity of U as a function of U_{*b} for a constant q noted in the present experiments and shown in figure 3 is also limited to the range of U and corresponding range of U_{*b} where major changes in the bed configuration occur.

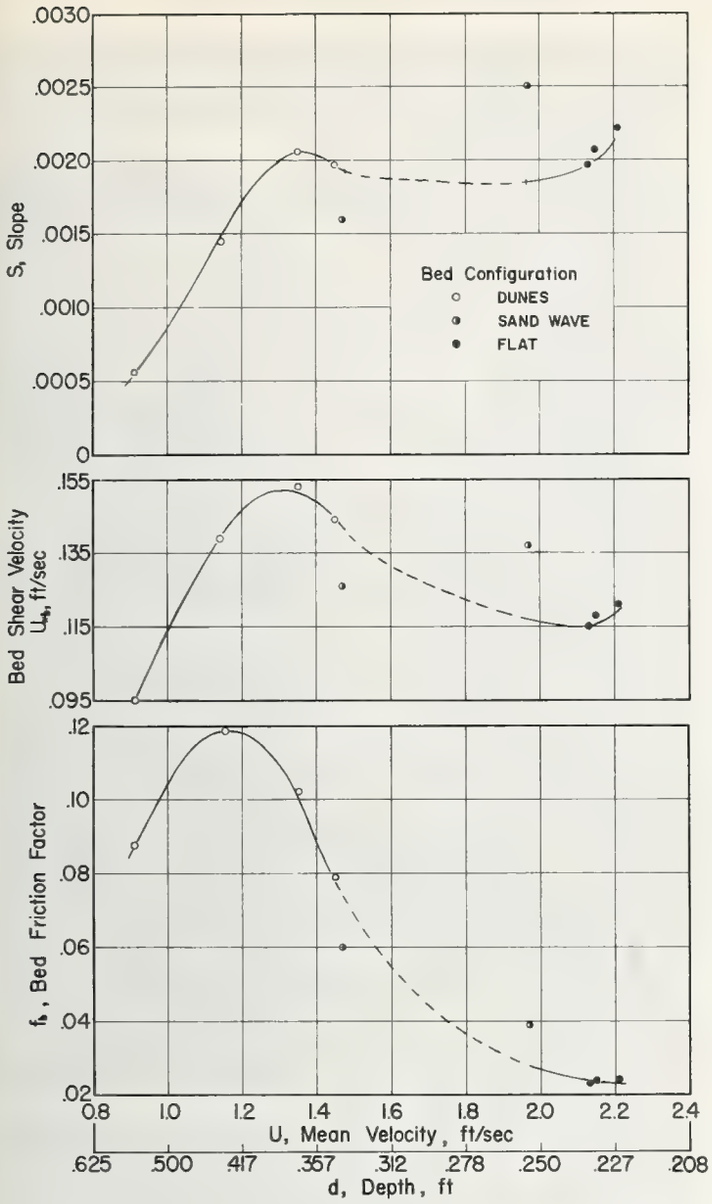

FIGURE 3. — Variation of slope, bed shear velocity, and bed friction factor with mean velocity and depth for constant-discharge experiments ($q = 0.50$ c.f.s. per foot).

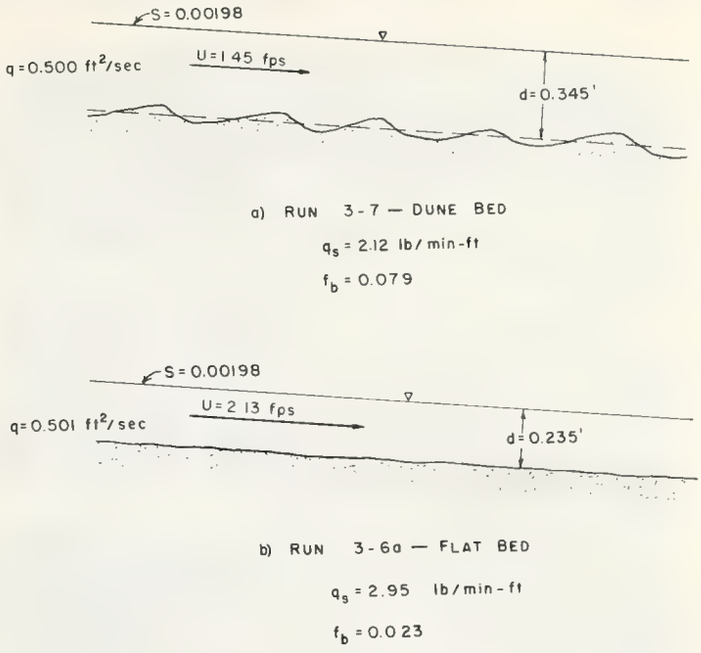

FIGURE 4. — Schematic diagram of two different equilibrium flows with same slope and discharge.

Thus, for the laboratory stream, the velocity for a given discharge cannot, in general, be expressed as a single-valued function of slope or bed shear velocity. However, each of the quantities S, U_{*b}, and f_b can be expressed as a single-valued function of the velocity, or in the case of these constant discharge runs, as a function of the depth. This is consistent with Brooks' conclusion No. 3. However, for velocities between 1.45 and 2.1 f.p.s., the nature of the variations of S, U_{*b}, and f_b is obscured by the sand waves that formed.

The bed shear velocity, U_{*b}, varied from a minimum of 0.095 f.p.s. to a maximum of 0.153 f.p.s., a factor of only 1.61. The maximum f_b was over five times greater than the minimum, and the slope varied by a factor of approximately four in the velocity range investigated. This comparison supports Vanoni and Brooks' conclusion No. 4g that U_{*b} is the least sensitive parameter that can be used to determine the hydraulic and transport characteristics of a flow.

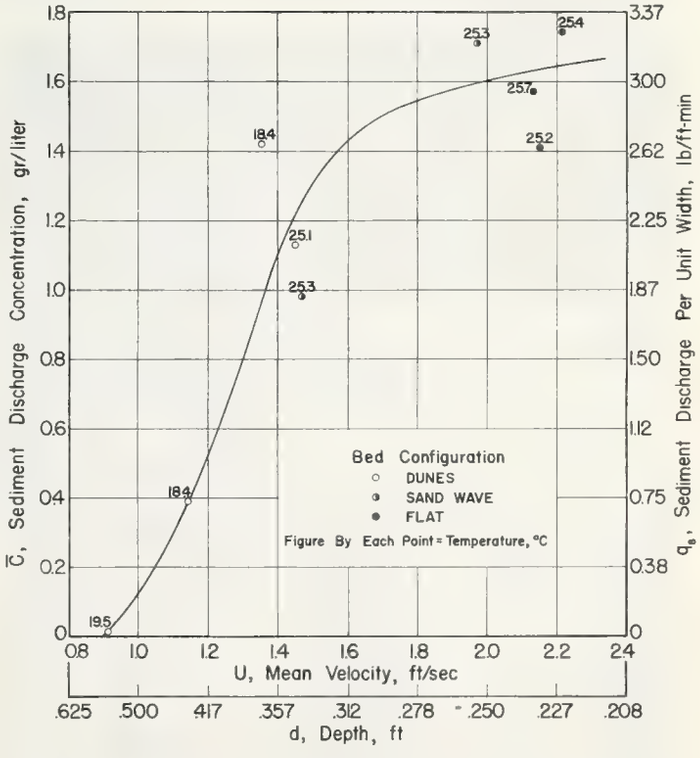

FIGURE 5. — Variation of sediment discharge concentration and sediment transport rate with mean velocity and depth for constant discharge experiments ($q = 0.50$ c.f.s. per foot).

Sediment Transport Rate

In figure 5 values of sediment discharge concentration, $C=q_s/q$, are plotted against mean velocity. Corresponding values of the sediment discharge per unit width, q_s, are shown on the ordinate scale at the right of the figure. The water temperature during the run is noted by each point.

After run 3–4 was completed, a heater was installed on the flume to control the water temperature. For run 3–5 and successive runs the water temperature was maintained at approximately 25° C. Runs 3–1, 3–2, and 3–4, performed before the heater was installed, had somewhat lower temperatures.

This temperature change is no doubt responsible for the apparent discontinuity in the velocity-sediment concentration relation between run 3–2 (U=1.35 f.p.s., C=1.42 gm./l., T=18.4° C.) and run 3–7 (U=1.45 f.p.s., C=1.13 gm./l., T=25.1° C.). The results of experiments with fine sand by Nomicos reported by Vanoni and Brooks (*19*) showed, for a constant discharge, that C and q_s generally decrease with increasing temperature. The drop in C between run 3–2 and run 3–7 is of the magnitude that Nomicos' experiments would indicate. Therefore, to put all transport data on a quantitatively comparable basis, the concentrations and sediment discharges for the runs at the three lowest velocities should be reduced about 25 to 35 percent.

With this adjustment (not shown), figure 5 indicates that C and q_s are single-valued functions of velocity for this constant discharge and that as the velocity is increased, these quantities also increase. This is consistent with Brooks' conclusion No. 4a that for a constant discharge an increase in q_s requires a decrease in d. As implied in Brooks' conclusion No. 4d, the high values of f_b are generally associated with the low values of q_s; however, the *maximum* f_b was not observed for the run with the lowest q_s, because of the slight initial rise of the f_b versus U curve (fig. 3).

The slight decrease in the geometric mean size of the sand that occurred between the time of Vanoni and Brooks' (*19*) experiments and the present investigation (see fig. 1) had a marked effect on the sediment transport rate, as may be illustrated by the comparison of Brooks and Vanoni's run 2–2 and run 3–6a of the present investigation. Table 3 gives a summary of the data for each of these runs, along with the transport rate of the individual sieve fractions and the cumulative transport rates of the material coarser than each sieve size. These two runs had identical velocities and depths, and temperatures that were nearly the same. For the material coarser than the 0.124 mm.-sieve the differences in the transport rates of the individual sieve fractions are insignificant in view of the temperature difference between the two runs, but for the finer sieve fractions the differences are striking. For each of these finer sieve fractions, significantly more material was transported in run 2–2 than in run 3–6a. This was because of the greater availability of fine material in the bed sand of run 2–2, as shown in the comparison of the sieve analyses of the bed materials in figure 1. The geometric mean sieve diameter, D_g, of the transported material of run 2–2 is accordingly much smaller, and the geometric standard deviation σ_g is larger, due to the extension of the frequency distribution toward the smaller sizes.

These two experiments point out the important effect the sand size distribution has on the sediment transport rate of the flow, and thus the necessity of accurately knowing the size distribution of both bed material and the load in analyzing, interpreting, and comparing the transport data for sediment-laden flows.

TABLE 3.—*Transport rate of individual sieve fractions*

Sieve opening (mm.)	Run 2–2 [Vanoni and Brooks (*19*)]: U=2.13 f.p.s.; d=0.233 ft.; D_g=0.095 mm.; σ_g=1.60; f_b=0.024; T=23.5° C.[1]			Run 3–6a U=2.13 f.p.s.; d=0.235 ft.; D_g=0.127 mm.; σ_g=1.41; f_b=0.023; T=25.7° C.[1]		
	Percent retained	Sediment discharge of sieve fraction	Cumulative discharge of coarser material	Percent retained	Sediment discharge of sieve fraction	Cumulative discharge of coarser material
		Lb./min.-ft.	*Lb./min.-ft.*		*Lb./min.-ft.*	*Lb./min.-ft.*
0.246	0.91	0.04	0.04	2.80	0.08	0.08
0.208	2.28	.11	.15	4.91	.14	.22
0.175	4.02	.19	.34	7.61	.22	.44
0.147	10.79	.50	.84	18.76	.55	.99
0.124	11.93	.55	1.39	17.87	.53	1.52
0.104	20.22	.94	2.33	23.15	.68	2.20
0.088	8.04	.37	2.70	10.41	.31	2.51
0.074	15.61	.73	3.43	9.42	.28	2.79
0.061	9.39	.44	3.87	2.30	.07	2.86
0.053	4.47	.21	4.08	1.89	.06	2.92
0.043	6.42	.30	4.38	.67	.02	2.94
Pan	5.92	.28	4.66	.21	.01	2.95
Sum	100.00	4.66		100.00	2.95	

[1] U = velocity; d = depth; D_g and σ_g = analysis of sediment load; f_b = bed friction factor; and T = temperature.

Remarks on Stream Morphology

In view of recent laboratory results (*1, 2, 10, 11, 12, 15, 16, 17, 18, 19, 21*), one may well ask which stream variables are functions of which other variables. For simple physical systems there is no confusion. For example, for a simple pendulum the length L may be considered a function of the period T and the acceleration of gravity g; but on the other hand T may just as well be considered a function of L and g. It makes no difference, because the relation is unique either way. However, if in a complex system there are many variables with several interlocking relationships, involving multiple-valued functions, the analysis may easily become confused. Selection of some variables, as independent and others as dependent, then becomes very useful for avoiding confusion and clarifying what functional relations are required.

As used here the term "independent" means that which is imposed on the system externally, and "dependent" means that which results when equilibrium is reached. "Unique" means that there is only one set of values of the dependent variables corresponding to the imposed values of the independent variables, whereas "multiple-valued" means that there may be two or more sets of values of the dependent variables and that there is *no basis for selecting one or the other purely on the basis of the chosen independent variables.*

A frequent error in applying dimensional analysis to sediment transport in streams is the failure to recognize all the variables and all the interdependencies between them simultaneously. For a most elementary alluvial stream flowing with uniform depth and velocity, the pertinent variables are as follows:

Q=discharge of water;
Q_s=discharge of sediment (of sizes found in the bed);
b=width;
d=mean depth;
r=hydraulic radius;
U=mean velocity;
S=slope of energy line=slope of water surface;
f=Darcy friction factor;
ν=kinematic viscosity of fluid (including any change due to suspended clays, etc.);
ρ=mass density of fluid;
ρ_s=mass density of sediment particles;
D_g=geometric mean size of bed material;[4]
σ_g=geometric standard deviation of sizes of bed material;[4]
w=geometric mean settling velocity of particles;[5]
g=acceleration due to gravity.

The above variables are interrelated in several ways, as follows:

1. Continuity:	$Q=Ubd$
2. Definition of f:	$U=\sqrt{\frac{8g}{f}}\sqrt{rS}$
3. Roughness function:	Einstein and Barbarossa (*9*) Shen (*15*)
4. Sediment transport function:	Brown (*3*) Chien (*4*) Vanoni, Brooks, and Kennedy (*20*)
5. Width-depth relation:	Leopold and Maddock (*13*); for flumes, b=constant
6. Hydraulic radius-depth relation:	$r=d$ for wide channels; r/d depends on shape of cross section in general.

The third relation predicts the roughness, including the effect of bed forms. The fourth gives sediment transport in relation to the other variables. Since there are no universally accepted formulations of relations 3 and 4 above, some general references are given instead of explicit equations. In the field, the width and shape of the cross section that govern relations 5 and 6 are self-adjusting to the variables listed above, plus some other geomorphic factors which are not listed above, such as bank resistance, hydrologic flow regime, and geologic history.

Since there are 15 basic variables and 6 dependencies, there are 9 independent and 6 dependent variables. In a laboratory flume (or canal) with fixed walls, there are only 5 dependent variables, because width is predetermined by the channel geometry and does not depend on the flow; i.e, we delete relation 5 above. Therefore, if 9 variables (10 in flumes) are selected or specified, the other 6 variables (5 in flumes) should be physically determined by the system. In flume experiments this is found to be the case, but for rivers the relations are not so clear because it is not possible to control the variables at will for test purposes. In

[4] Logarithmically normal distributions are implied for the diameter and fall velocity of the sediment particles comprising the bed, as is frequently the case for natural streams.

[5] If all particles were spherical, w would be determined by other variables on the list (D_g, ρ_s, ρ, ν, g); for nonspherical particles an indication of the shape is thus given by w and the other variables considered together.

TABLE 4.—*Choices of independent and dependent variables for steady equilibrium flow in alluvial streams*

System	Independent variables		Dependent variables (not all combinations listed)	Functional relations are:	
	Properties of fluid, sediment, gravity, etc.	Characteristics of flow system (not all combinations listed)		Unique	Multiple-valued for some ranges
Flumes	$\nu,\rho,\rho_s,D_g,\sigma_g,w,g$	Q,d,b	Q_s,U,r,S,f	x	
	do	Q,Q_s,b	d,r,U,S,f	x	
	do	U,d,b	Q,Q_s,r,S,f	x	
	do	d,S,b	Q,Q_s,r,U,f		x
	do	r,S,b	Q,Q_s,d,U,f		x
	do	Q,S,b	Q_s,d,r,U,f		x
Field:	$\nu,\rho,\rho_s,D_g,\sigma_g,w,g$	Q,d	Q_s,b,r,U,S,f	x	
Short term	do	d,S	Q,Q_s,b,r,U,f		x
	do	r,S	Q,Q_s,b,d,U,f		x
	do	Q,S	Q_s,b,d,r,U,f		x
Long term ("graded stream")	ν,ρ,ρ_s,g	Q,Q_s	$b,d,r,U,S,f,D_g,\sigma_g,w$	x	
Very long term	ν,ρ,ρ_s,g, geology	Climate, man-made works	$Q,Q_s,b,d,r,U,S,f,D_g,\sigma_g,w$	x	

table 4 an attempt has been made to summarize experience to date in flumes and to indicate the implications for the field. Various alternate classifications of the variables are found by reading across on any line of the table.

As indicated in table 4 some choices of independent variables do not define a unique result. Thus, for the six cases with "x" in the "multiple-valued" column, the choice of independent variables is not really correct. For example, as shown in the preceding sections, specifying discharge Q and slope S in laboratory flumes will not always define uniquely the flow condition of the stream. Instead there may be several possible flow conditions, including one with low velocity, high depth, high roughness, low transport, and a dune bed, and another with high velocity, low depth, low roughness, high transport, and a flat bed. Each flow can easily be generated in a flume and will remain stable over a long period of time, because there are other constraints such as depth regulation, continuity of sediment transport, and continuity of the energy grade line.

One might be tempted to say that the problem should be formulated in two parts: one for dune bed cases, one for flat bed cases, etc. Since within each regime there will probably then be no multiplicity, one might think that the nonuniqueness problem could thus be avoided. But experience shows that the bed form itself is strongly related to the velocity and cannot be deduced solely from the independent variables listed for "multiple-valued" cases in table 4; thus we are back where we started from with an indeterminant relation.

The slope has very commonly been thought of as an independent variable, but flume experience quickly teaches otherwise. In a typical flume experiment, one sets the flume to some estimated slope and then waits to see if the energy gradient of the established flow matches that of the apparatus; if it does not, then the *apparatus* slope must be adjusted to the *energy* slope. Alternatively, the *energy* slope can sometimes be made to match that of the flume by successive adjustments of the discharge or depth in a closed circuit flume, or the discharge and the sediment feed rate in an open circuit flume.

In the field the identification of independent and dependent variables, or cause and effect, is clouded by long-term intercorrelations naturally established; parameters cannot be varied at will but have certain values produced by the system itself. For example, at a given time and place one may find by sampling that the bed sand for a given stream has a certain mean diameter D_g, which is indeed "imposed" on that stream for any conditions occurring in the near future ("short term"). But from a broader point of view, it must be recognized that the stream itself established the D_g value through years of transport, sorting, and abrasion. Therefore, for the short term the bed sand size D_g is an independent variable, but on the geologic time scale D_g is certainly a dependent variable. Similarly, the slope of an alluvial river changes very slowly, but at any time the slope is indeed a slope that is compatible with the geology and general hydrologic flow regime because these factors over geologic time have established the slope.

The time scale under consideration is also important in classifying Q and Q_s as dependent or independent variables. Streams are seldom, if ever, in a steady state (because of finite time required to change bed forms and depth), and transitory adjustments are accomplished by storage of water and sediment. Water storage is relatively short (hours and days) and occurs simply by a rise of river stage or overbank flooding; sediment storage (plus or minus) occurs by deposition or scour. Over the short term Q_s, the sediment transport rate, in any reach, will depend on Q and the associated value of d re-

quired to give continuity in the water surface and energy grade line at the ends of the reach at any moment. Thus, for the short term, Q_s may be considered a dependent variable, with departures of the sediment inflow from the equilibrium transport rate being absorbed in temporary storage (for perhaps months or years).

However, in the long term the river must assume a profile and other characteristics for which, on the average, the inflows of water and sediment equal the outflows; consequently, for this case (called a "graded stream" by Mackin (*14*)), Q and Q_s are listed in table 4 as the independent variables. In this instance, Q and Q_s need not necessarily be particular single values but also may be considered to represent related sequences of values with fixed statistical characteristics. For the very long term, streams respond to the geology of the region, the climate, and the properties of matter. From this point of view, every characteristic of a river is dependent on something else.

Even for the short term in the field the occurrence of discontinuous rating curves for streams (*5*, *7*) suggests that depth and slope are not always sufficient to define the instantaneous discharge. In fact, the authors are of the opinion that alluvial rivers naturally evolve into the ranges of variables (slope, depth, velocity) where relations are multiple-valued due to changes in bed form. If generally so, this is indeed fortunate, for alluvial streams respond to flood flows by reducing their hydraulic roughness as well as by increasing stage. If the roughness did not change, then far greater increases in stage would be required to allow passage of floods! Similarly, high roughness with low flows is beneficial, since it provides adequate depth for navigation with as little flow as possible.

Until recently sediment transport equations have almost universally been built around the idea that bed shear is an essential parameter. [See summary as of 1950 in (*3*).] Yet table 4 shows that r and S, and hence the shear $\tau=\rho g r S$, do not uniquely determine the flow or the sediment transport rate. The experiments reported in the preceding sections show that the shear velocity changes least of any variables, and thus is certainly a poor starting place for a transport calculation. Colby (*6*) has recently suggested that stream velocity is a far better variable with which to correlate sediment transport.

In this regard, Einstein (*8*) and Laursen (*12*) have recently used τ', the grain resistance, in place of τ in formulas for sediment transport. But τ' is given by

$$\tau'=\frac{f'}{8}\rho U^2$$

where f' is a friction factor expected for flow over a flat bed, and may be taken from the pipe-friction diagram or the Prandtl-Kármán resistance equations (in the ways indicated by Einstein and Laursen). Since f' is not related to the total friction factor f for the actual flow, τ' is not a measure of the total shear but is for all practical purposes a parameter reflecting the velocity. Thus, there has been a subtle shift toward computing sediment transport from velocity rather than from total shear stress.

Summary and Conclusions

The present experiments corroborate, for this sand and unit discharge, Brooks' conclusion No. 4b that for a given slope and discharge two different depths of flow are possible in a laboratory flume. This conclusion is valid, however, only for slopes in the range where major changes in the bed configuration occur. It was also found that for runs at a constant discharge, the velocity cannot in general be expressed as a single-valued function of shear velocity. However, in the range investigated, if the velocity is used as the independent variable, then the slope, bed friction factor, and bed shear velocity are uniquely determined; i.e., they are single-valued functions of velocity.

For these runs at a constant discharge, an increase in the sediment discharge required an increase in the velocity and a corresponding decrease in the depth; this is in agreement with Brooks' conclusion No. 4a. The results of these experiments qualify Brooks' conclusion No. 4d that the largest bed friction factors are associated with the smallest sediment transport rates, because, with increasing velocity, the bed friction factor first increased slightly, then decreased. However, the sediment discharge increased steadily with increasing velocity.

A comparison of the results from the current experiments and the earlier experiments of Vanoni and Brooks indicated that even very slight changes in the size distribution of the bed material can have a significant effect on the sediment discharge rate and the size distribution of the transported material.

Table 4 shows various possible choices of independent and dependent variables for describing alluvial streams in laboratory flumes and in the field, and indicates which relations are unique and which are non-unique. In the field it is necessary to differentiate between long term and short term points of view. Whereas sediment discharge, Q_s, may be considered one of the *independent* variables for a particular reach of river in the long term, it is more likely to be a dependent variable for the short term. A graded river adjusts itself to the flow of water and sediment in the long term, but during short

periods the sediment inflow to a reach can be far out of balance with the equilibrium transport rate, resulting in short term sediment storage.

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SPIRAL MOTION, SEDIMENT TRANSPORT, AND RIVER DEVELOPMENT

[Paper No. 38]

By WILLIAM F. TANNER, *associate professor, Department of Geology, Florida State University*

Abstract

In fluids, spiral motion is common.

Turbulence behind a falling grain arises from three vortex centers, spaced around the perimeter at about 120° intervals, which shed vortices in sequence. Shedding of an individual vortex tends to rotate the grain in a predictable direction, which, in turn, produces a spiral path.

At Froude numbers above one, shallow water passing an obstacle exhibits a vortex trail, having a spiral nature (down on the outside, up on the inside). Outside of each line of vortices, a standing spiral wave may develop (also down on the outside, up on the inside). Two waves make an ogive behind the obstruction. A roughness element on the channel wall creates one wave. Two waves, from opposite walls, cross near mid-channel in the "rooster tail." In shallow water, spiral waves arise at low velocities, and erode the bed, without any vortex trail being present.

A larger spiral overturn (helical, or secondary flow) operates to divert the main stream of water in the direction of the slow (i.e., rising) side. In deep alluvial streams, this process may be responsible for meandering; in wide, shallow streams, it may induce a system of higher order spirals that, taken together, account for the braided pattern.

Introduction

Spiral motion in fluids is common. It appears to be important in a variety of processes in the atmosphere and in the hydrosphere (*16*). It definitely plays a part in the shaping of stream channels and water current beds, and it may be significant in the transportation of sediment.

Above a certain transition zone (commonly defined as near R=3,000), fluid flow is generally turbulent. The rotational motion within this turbulence can be combined with translation, or straight-line movement, to produce a spiral pattern of sorts and, hence, provide a basis for the general statement that spiral motion in fluids is common. It is the purpose of the present paper, however, to present more specific observations of spiral flow.

Single Grain Studies

Individual grains falling through a fluid medium have been the subject of study for a century or more (*2*, *7*). Terminal velocities have been established for certain single grains, under specified conditions. More refined experiments have shown that this terminal velocity depends greatly on the shape and surface of the grain. Rarely has it been observed, however, that even with a single smooth spherical grain, settling through a stilled medium, it is difficult to obtain straight-line motion.

Investigation of behavior of falling grains can be carried out conveniently by means of "scaled up" models, which are large enough to permit detailed examination of flow patterns. For example, we may wish to use a model, 4 cm. in diameter, to represent a quartz grain 0.4 mm. in diameter. The model ratio of length is thus specified as L=100. Inasmuch as the model and the prototype both operate in the earth's gravitational field, the acceleration of gravity is the same for both and the model ratio of gravitational acceleration is unity. This, however, specifies a rigid relation between the dimensions length and time, inasmuch as acceleration is a length per time squared. Substituting 100 for L, and solving the equation (LT^{-2}=1), we determine a model ratio of time as T=10. The scaled-up model must be 100 times as wide as the prototype and must take 10 times as long to settle an equal distance. Settling velocities can be controlled by altering the mass of the model. In the experiments described here a Ping-pong ball nearly full of water has been used as the model (fig. 1).

One additional fact must be stated: the Reynolds regime for the model must be similar to that of the prototype. Inasmuch as length has been specified (arbitrarily), time has been derived from length and mass derived from velocity; only fluid viscosity remains to be determined. The experimenter is free to vary fluid viscosity as he wills, provided he does not alter settling velocities beyond his ability to correct by means of adjusting mass. A wide range of fluid viscosities, from 1,600 cp. to 1 cp. (centipoises), has been used.

The model that meets all of these requirements has interesting behavior indeed. For Reynolds regimes in the range of turbulence, no straight-line fall has been produced. Straight-line fall has been obtained only with very heavy metals (i.e., steel) moving through very viscous liquids (i.e., 1,000–1,600 cp.). Reynolds numbers for these models have been R=1 or less. Laminar motion of fluid elements has been established by use of dye patches on the surface of the model grain.

At Reynolds numbers in the neighborhood of R=10, turbulence appears behind the falling model. (If we wish to maintain the fiction that R=3,000 is an unbreakable limit, we must introduce a factor of roughly 1,000, which we might identify as a "grain fudge factor," into the Reynolds number definition.) This turbulence can be studied by dye tracer methods. Turbulence has occurred in all models that have been run in the range of R=10 to R=1,000.

Three vortex centers commonly appear on the trailing side of the falling model. These are spaced around the periphery of the model at about 120° intervals. Vortices are shed, in sequence, from each of the three centers. Addition of roughness elements to the model surface tends to hasten the development and increase the size of the vortices.

The Kármán vortex trail (*7*), commonly observed in the wake of a boulder protruding above the surface of a rapidly flowing stream, arises from two vortex centers that exist in the plane of the water surface. The turbulence pattern behind the falling model is analogous to the water-surface vortex trail, except that it is three-dimensional (and has three centers) instead of two-dimensional (and having two centers).

The motion in the vortices is such that a filament of fluid directly behind the falling model tends to move in the same direction as the model. In the water-surface Kármán vortex trail, behind an obstruction to flow, this filament actually may move in an upstream direction. Development and detachment of a single vortex,

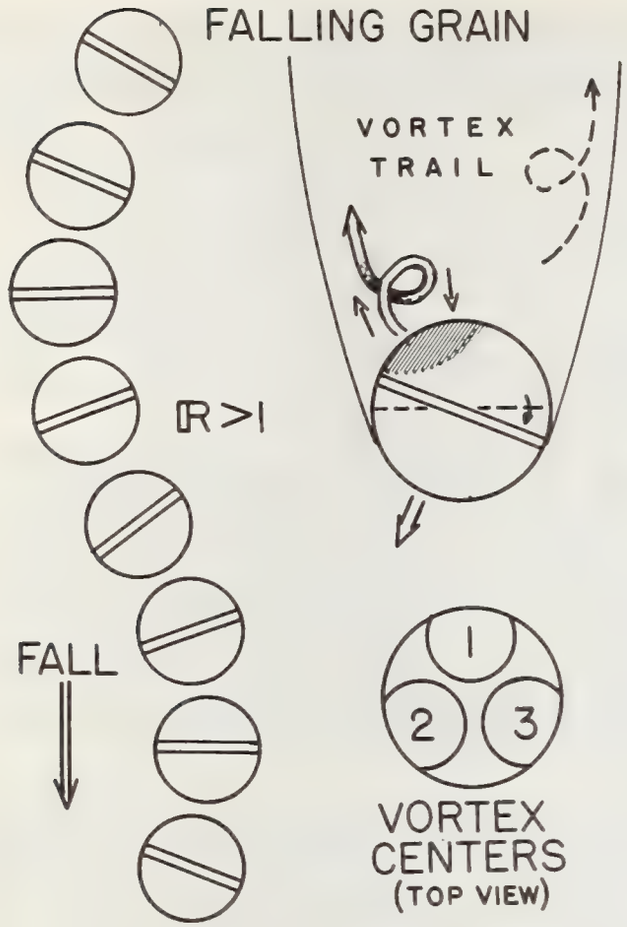


FIGURE 1. — Settling velocities and regimes.

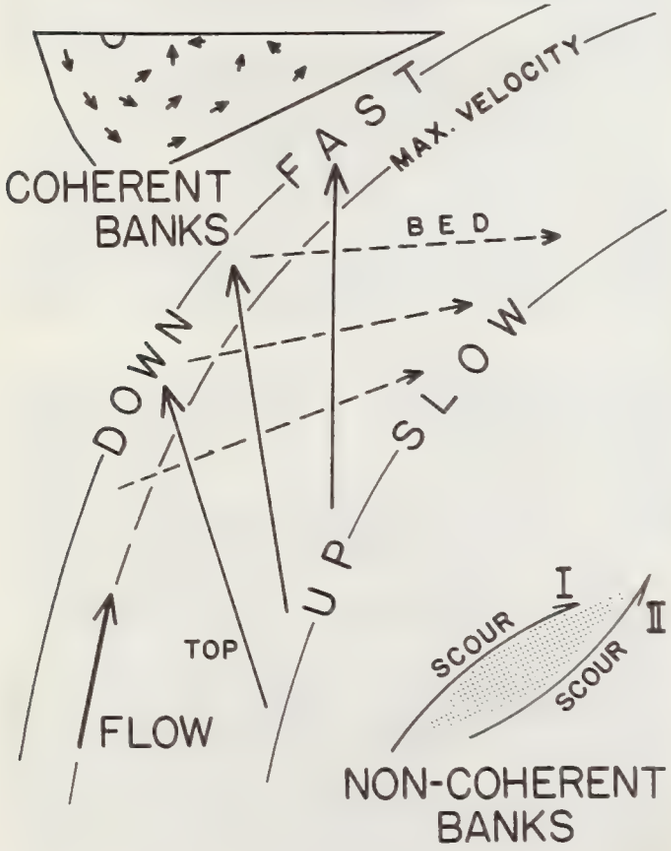
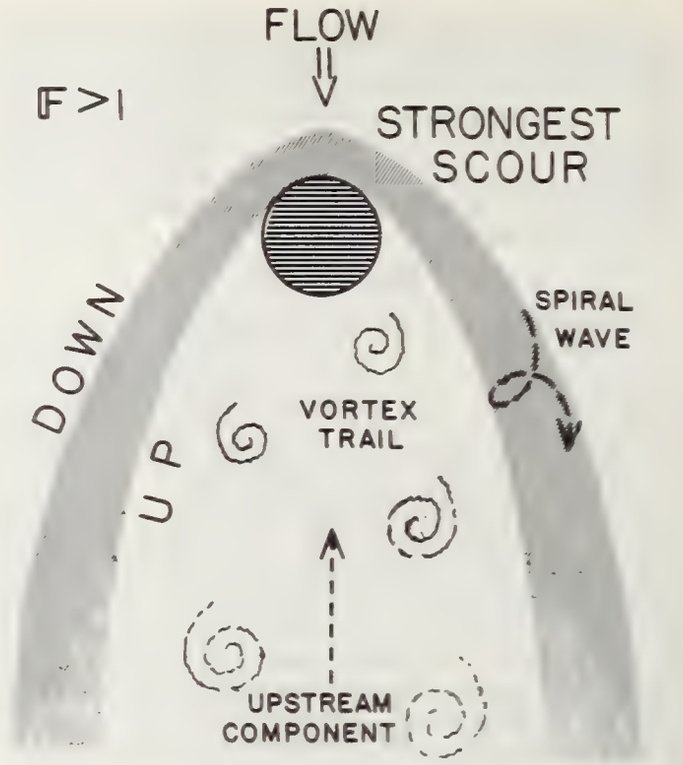


FIGURE 3. — Directions of scour patterns.



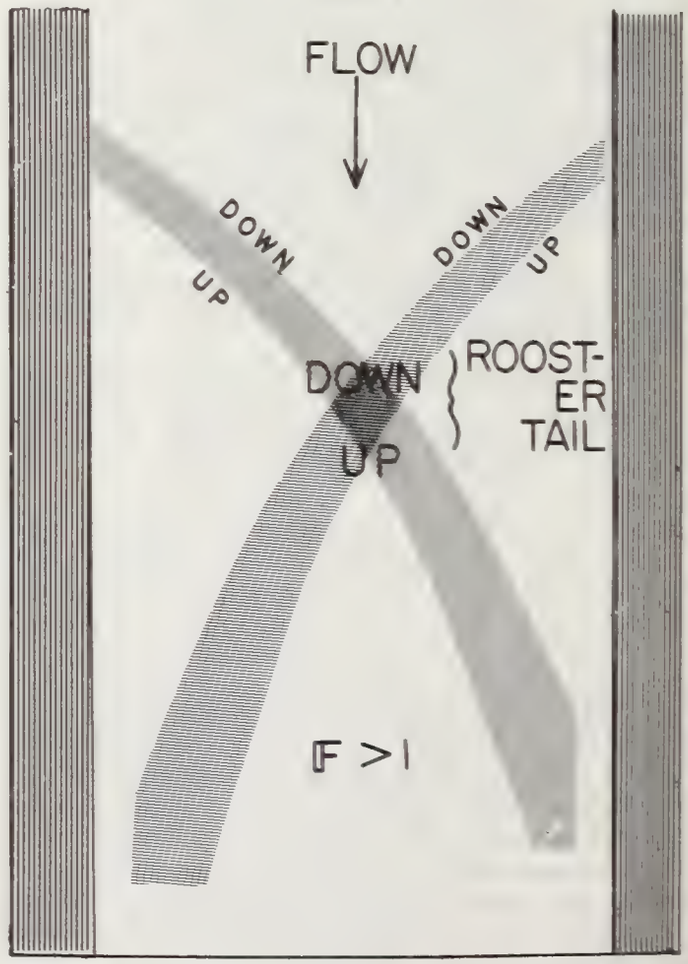


FIGURE 2. — Vortex and wave patterns of streamflow for Froude numbers above 1.

located to one side of the downward-moving particle, tends to rotate the model and to modify its direction of fall. When a second vortex develops, the rotation and fall direction are modified again. Continuous shedding of vortices combines these modifications into smooth movements, which are seen by an observer as a spinning effect and a spiraling effect.

The spiral can be noted from the side, but is most obvious when seen from above. The spin can be made clear by painting a bright equatorial band on the model grain.

Coarse to medium sand-sized particles, if spherical and smooth, settle with Reynolds numbers in the neighborhood of $R=10$ or $R=100$ and spiral and spin. Dye studies show turbulence in their wakes. If the grain shape is altered from a sphere and the surface is roughened suitably, the spiral path can be obtained with fine or very fine sand. Straight-line, or laminar-flow, settling appears to be limited under natural conditions to silt-sized particles.

Because the two settling mechanisms are so vastly different, a single settling velocity curve cannot be constructed for all common sediment sizes. Settling velocities of coarser particles are approximated by the impact law; for finer particles, by Stokes' law, as is well known.

It is possible that this difference between the two size ranges is important in understanding the relative ease with which the coarser grains are placed in motion. Turbulence at the bottom, with attendant momentary increased velocities, might be significant in the scour mechanism, in a way analogous to the spinning and spiraling of the falling model. Bed velocities that would be needed would be approximately 1 cm./sec. for coarse sand sizes and 10 cm./sec. for fine sand sizes. Bed velocities in this range should be fairly common.

The Spiral Wave

At Froude numbers above one, shallow water flowing past an obstacle exhibits the Kármán vortex trail. Motion within individual vortices is both down and downstream near the boundary between the vortex trail and the main flow, and up and upstream near the boundary between the two sets of vortex elements. This pattern is responsible for the pointed deposits of sand which accumulate, in some cases, behind cobbles and boulders (fig. 2).

Outside of the vortex trail, a standing spiral wave may develop. In a plan view, the wave is gently convex. Two standing waves that arise from a single obstruction combine to make an ogive pattern with its tip near the upstream edge of the boulder. The motion in each wave is down on the outside (convex side), up on the inside (concave side). Because of the general downstream motion, flow of a single parcel of water follows a spiral path. Maximum local velocity should be found in the strip of down flow. Scour can be expected, in favorable instances, along this strip (*17*). Maximum scour should develop near the apex of the ogive, immediately upstream from the obstruction. Scour of this type has been observed in natural high-gradient streams and in flumes and small models. In some cases, the spiral wave develops without the benefit of any obvious obstruction; the precise mechanism, in this instance, is not clear. In shallow water, it is not necessary that a vortex trail be present. The latter is a Reynolds regime effect; the spiral wave a Froude regime manifestation. Vortex trails without spiral standing waves appear when the Froude number is less than 1, but the Reynolds number is on the order of $R=100$ to $R=1{,}000$.

A roughness element on a channel wall may generate a single-spiral standing wave, which forms half of the ogive pattern, and vanishes in the downstream direction. Two roughness elements, on opposite channel walls, may give rise to two spiral waves that cross in midstream. At the junction of these waves, a spout, or breaking wave or "rooster tail," may appear (*4*). In the models that have been studied to date, the geometry of the rooster tail appears to have been related to the width-depth ratio. The characteristic width that is used is not necessarily the channel width, inasmuch as spiral waves may be produced by adjacent roughness elements on the channel bed (i.e., two adjacent cobbles or boulders).

The rooster tail is commonly associated, over sand beds, with violently breaking antidunes and patches of turbulent suspended sand. Individual grains pass through these patches with momentary delays.

In water 1 cm. deep, a velocity of about 30 cm./sec. is enough to initiate a spiral wave, which may, in turn, erode the bed. The critical point at which both vortices and the spiral wave may appear is roughly defined by $F=1$ and $R=10$; this occurs in water where the characteristic length is about 0.35 cm. and the velocity is about 50 cm./sec.

The Larger Spiral

Spiral flow on a larger scale, at Froude numbers less than $F=1$, and without benefit of a standing wave, is common in moving fluids. Retardation of part of a moving parcel of air or water, perhaps because of roughness, results in veering and overturning (*1*). The current tends to bend toward the decelerated side. The slower part of the fluid rises, and the faster part sinks [also note Shen (*9*)]. A general spiral develops, which is damped out in a map distance equal to some tens of times the water depth (*14*). The velocity gradient, from the

bed toward the thread of maximum current motion, is steep on the fast, descending, outer side, and is gentle on the slow, ascending, inner side (*5*).

The various effects that have been listed here (deceleration and acceleration, veering, overturning) appear to arise simultaneously. They appear in initially straight streams when the Froude number is less than $F=1$ and the Reynolds number is greater than $R=100$; it is possible that they also appear at somewhat lower Reynolds numbers. These effects do *not* depend on the presence or absence of mobile sediments of any kind (*10*).

Inasmuch as the descending part of the current is significantly faster than the ascending part and inasmuch as the velocity gradient is much steeper where the water descends, active scour occurs here. This scour is directed both downward, toward the channel bed, and laterally, toward the channel wall (fig 3).

Where a spiral cell begins to dissipate, the current is *not* flowing directly down the gravity slope. This fact predisposes the next ensuing spiral cell to veer and overturn, in such a way that direct downslope motion will be established. In other words, spiral flow alternates in direction. This oscillation accounts for the offset pattern of scour holes in a straight channel.

Lateral scour, at each scour site, is the crux of the meandering process. Ultimately a "most efficient" curvature will develop, after which the meander geometry stays within certain limits but "slides" down the length of the stream valley.

Actual circulation within a stream channel is much more complex than stated above. Secondary and higher order cells complicate the picture greatly (*3, 5*). However, the basic flow pattern is as indicated, and it has been confirmed over a period of many years by various workers under a variety of conditions. The chief remaining question is, which comes first, the spiral cell or the meander loop? This question must be answered in favor of the spiral cell, which is frequently observed in straight channels.

Where streambanks are composed of more-or-less coherent materials (such as sand with a small admixture of clay or organic matter), a deep channel is maintained and meandering arises unless velocities are too low. Where streambanks are composed of noncoherent materials (loose sand; gravel), a deep channel cannot be maintained, and the width-depth ratio of the water becomes fairly great. Under these conditions a single spiral cell occupies only a relatively small part of the total channel width and induces second and higher order cells in nearby parts of the current (*6, 11, 12*). Any two adjacent cells have opposite senses of spiral and curvature.

If two cells face each other (concave facing concave), an elliptical, or diamond-shaped, area of rather slow, rising water appears between them (*13*). This area is a site of potential deposition; in it, a diamond-shaped bar can be built. If two adjacent cells are back-to-back (convex against convex), a narrow, swift, descending current appears, resulting in scouring of the streambed. Because of the time lag in the process of inducing higher order cells, adjacent spirals are generally staggered rather than being located precisely face-to-face or back-to-back. This staggering of the diamond-shaped bar pattern produces the phenomenon known as braiding.

Other Spirals

Spiral flow has been observed in connection with dust devils, sand dunes, tidal flat ripple marks (*15*), and possibly sand "waves" on the sea floor. It also occurs under a variety of artificial conditions, including Reiner's "teapot effect" (*8, p. 26*), the bath tub vortex, and in surface tension and other models. Some of these effects appear to be significant in the transportation of sediment and the shaping of land forms, but are not of interest in connection with river studies.

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CHANNEL DESIGN FOR MODIFIED SEDIMENT REGIME CONDITIONS ON THE ARKANSAS RIVER

[Paper No. 39]

By EDWARD B. MADDEN, *hydraulic engineer, U.S. Army Engineer Division, Dallas*

Synopsis

The Arkansas River in Arkansas and eastern Oklahoma is being developed for navigation, flood control, hydroelectric power generation, and other uses by means of a series of navigation locks and dams and upstream storage reservoirs. Entrapment of sediment in the upstream storage reservoirs and in the larger ones of the main stem navigation pools will reduce the present 100-million-ton-per-year sediment load on the lower river by about 90 percent and will induce extensive degradation of the streambed downstream from the dams. The project plan provides for taking advantage of the degradation by spacing out and reducing the number of low-lift navigation locks and dams from that which would be required to match the natural river profile. The navigation channel in the upstream ends of the pools, to be developed initially by dredging and contraction work in advance of natural degradation, was designed to conform to the modified regime conditions by application of relations correlating the channel geometry and slope, streambed composition, water discharge, sediment load capacity of the channel, and sediment supply.

Description of Project

The Arkansas River in Arkansas and eastern Oklahoma is being developed for navigation, flood control, hydroelectric power generation and other uses. The plan of development for the multiple-purpose project is shown on figure 1. Included in the project are a number of large storage reservoirs that serve or will serve variously for flood control, hydroelectric power, water supply for navigation, domestic and industrial water supply, and sediment detention.

The navigation feature of the project provides for a channel following the Verdigris River from Catoosa, Okla. (near Tulsa), downstream to the Arkansas River, thence along the Arkansas River downstream to the vicinity of Arkansas Post, Ark., from which point the route will follow a canal to the White River, then down the White River to the Mississippi River. The authorized minimum navigable depth is 9 feet over minimum widths of 250 feet in the Arkansas and White Rivers and 150 feet in the Verdigris. A depth of 12 feet is authorized for dredging in canals and on limiting shoals. The waterway will be canalized by a series of navigation locks and dams. Four of the navigation dams on the main stem will have moderately high heads ranging from 30 to 54 feet. Two of these will include provisions for hydroelectric power generation initially, and the other two will be so designed as to permit possible future addition of power facilities. The remaining navigation locks and dams will be relatively low-lift structures (generally 20 feet or less) with pools confined within the river channel. The low-lift dams will be gated and the gate sills will be at approximate streambed level, with the result that essentially open river conditions will prevail at the higher flows in excess of approximately half-bankfull capacity.

Stabilization and rectification of the rapidly shifting channel of the Arkansas River by means of stone and pile revetments and dikes is an additional major feature of the project. The stabilization and rectification work will

FIGURE 1. — General plan and profile of the multiple-purpose plan for the Arkansas River and tributaries in Arkansas and eastern Oklahoma.

extend from Short Mountain Lock and Dam to the vicinity of Arkansas Post and will be continuous throughout except in the larger main stem pools and in a number of short naturally stable reaches. The project design contemplates taking advantage of the expected degradation of the channel downstream from the larger main stem dams and provides for special measures for dredging and channel contraction in the upper ends of the navigation pools in the interest of increasing depths, so as to permit spacing dams farther apart than could otherwise be done. A description of the design procedures used to determine the degree of contraction and amount of dredging required to produce and maintain a channel of the required navigable depth and to conform to the modified regime conditions is the principal subject of this paper.

Construction Schedule

The entire Arkansas River multiple-purpose project is scheduled for completion in 1970. Construction of the low-lift navigation locks and dams, which will begin in 1964, will be carried on generally in upstream order so as to provide a completed navigation channel successively to Pine Bluff, Little Rock and Fort Smith, Ark., and the vicinity of Tulsa, Okla. Oologah, Pensacola, Fort Gibson, Tenkiller Ferry, Heyburn, Wister, Blue Mountain, and Nimrod Reservoirs are existing. Dardanelle, Keystone, Eufaula, and Markham Ferry Reservoirs are under construction, with filling of the pools expected to begin in 1964. The bank stabilization and channel rectification work is about 50 percent complete. The final phases of the contraction and dredging in the head of each pool will be completed in coordination with the completion of the next upstream navigation lock and dam. As will be brought out later, the sequence of construction of the reservoirs and the locks and dams is a major factor in the design for the contraction and dredging work.

Stream Characteristics

Streamflow

Average flows in the reach of the main stem of the Arkansas River to be traversed by the navigation route range from about 20,000 c.f.s. at Muskogee, Okla., to 42,000 at Little Rock. Representative maximum unregulated peak flows are 700,000 c.f.s. at Muskogee, 875,000 in the vicinity of Fort Smith, Ark., 720,000 at Dardanelle, and 700,000 at Little Rock. Regulation by the storage reservoirs will reduce these peaks to 560,000, 516,000, 492,000, and 448,000 c.f.s., respectively, the reductions ranging from 25 to 41 percent of the unregulated peaks. Minimum monthly unregulated flows downstream from the Canadian River range from 519 c.f.s. at Muskogee to 874 at Little Rock. Corresponding minimum monthly regulated flows will range from about 3,700 to 4,600 c.f.s. Bankfull channel capacities are about 160,000 c.f.s. in the vicinity of Fort Smith and 250,000 at and downstream from the vicinity of Dardanelle.

Slopes

Natural river slopes average about 1 foot per mile between Muskogee and Fort Smith, 0.9 between Fort Smith and Dardanelle, 0.8 between Dardanelle and Little Rock, 0.7 between Little Rock and Pine Bluff, and 0.6 between Pine Bluff and the mouth of the river.

Channel Dimensions

Natural channel widths in the pertinent part of the Arkansas River between Short Mountain Dam site and Arkansas Post range from about 800 feet at some localized naturally stable areas to 3,000 feet in a number of unstable "wild" reaches. Bank heights range from about 20 to 30 feet above mean low water. Depths in pools are generally from 5 to 20 feet below mean low water, and limiting depths on many crossing shoals are less than 3 feet at mean low water and 1 foot or less at extremely low stages.

Sediment

The large sediment load carried by the Arkansas River presents the major problem in the planning and design of the project. The natural suspended sediment load downstream from the mouth of the Canadian River ranges from 95 million tons per year at Short Mountain Dam site to 105 at Little Rock. About 24 percent of this is suspended bed-material load consisting of sand sizes contained in the streambed in appreciable quantities. About 50 percent of the suspended load is contributed by the Canadian River, 25 by the Arkansas River upstream from Tulsa, and the other 25 by the other tributaries in eastern Oklahoma and Arkansas. Bedload, or the material rolling or sliding generally in contact with the bed, is estimated, on the basis of calculations by bedload formulas and from some rough measurements, to be about 5 percent of the suspended bed material load. The streambed is composed predominantly of fine to medium sand, with smaller percentages of very fine sand and of material coarser than medium sand, including appreciable percentages of gravel ranging up to 1 or 2 inches in size.

Effect of Reservoirs on Sediment

As part of the design studies for the project, computations were made to estimate the amount of deposition in each of the reservoirs and in the pools of the moderate-head main stem dams and the sediment outflow from the dams. In effect, the loads were routed downstream through the pools and the open river reaches between pools, taking into account the loads contributed by the tributaries and scoured from the streambed and unstabilized banks of the open river reaches. This routing of flows was carried as far downstream as Dardanelle Dam and covered a period of 50 years. The computations indicated a sediment outflow of about 10 million tons per year from Dardanelle Dam, which is one-tenth the natural load transported past this site. About 8 million tons per year of the sediment outflow would consist of the silt clay fractions and 2 million tons per year would be sand. The details of the computation procedures are beyond the scope of this paper. However, in summary, the sediment deposition in and outflow from the large run-of-river mainstem pools were estimated from curves of relation between the detention time in the reservoirs and the ratio of sediment deposition to sediment inflow. These relationship curves were developed from data of existing reservoirs.

Channel Degradation

With such a large reduction in sediment load as that indicated by the studies described in the preceding paragraph, extensive degrading of the streambed can be expected to take place downstream from Dardanelle, Ozark, and Short Mountain Dams as the river attempts to replenish its sediment load by scour of the streambed. As the degradation proceeds downstream from each of these dams, the stream slope will flatten and the material in the streambed will coarsen, with the result that the transport capacity will reduce until a condition of equilibrium is established between the transport capacity of the channel and the reduced sediment outflow from the dams. Additional lowering of the streambed in some places will result from the shortening of the length of the river by the develop-

ment of artificial cutoffs and bend easing as part of the bank stabilization and channel rectification work. This shortening will amount to 23 miles between Arkansas Post and Dardanelle Dam and 7 miles between the head of Ozark Reservoir and Short Mountain Dam. The degradation not only will make available a large increase in head for generation of power at the main stem power dams but also will make it possible to increase the spacing between and reduce the number of the low-lift navigation dams from those that would be required to match the natural river profile. Natural degradation will progress slowly downstream and would be retarded and eventually arrested by the formation of an armouring layer of gravel on the bed surface, such as has been observed in degrading streams elsewhere and in sediment transport flume studies. Therefore, dredging and contraction work will be performed in the heads of the navigation pools in advance of the natural degradation, in order to provide a channel of the required navigable depth by the time the navigation locks and dams are completed in the separate sections of the river. The dredging will prevent the gravel armouring from inhibiting degradation. Any armouring that occurs after dredging will be beneficial in stabilizing the dredged channel and in preventing excessive degradation which could be detrimental to the locks and dams.

Channel Design Procedure

General

In establishing the damsites and pool elevations, it was necessary to compute the dimensions and bottom grade elevations of dredged channels in the upstream ends of the pools that would have the required navigable depth below the flat pool level and which could be maintained in equilibrium by a reasonable amount of contraction works. The start of navigation will be the most critical time for obtaining navigable depths. Therefore, the sediment load as modified by the reservoirs at that time will be the controlling equilibrium condition for channel design. Continued degradation after the initial dredging will improve navigable depths. The term "equilibrium" defines the condition under which, for a given discharge or sequence of discharges, the sediment transport capacity of a channel is equal to the sediment supply and neither scouring nor filling of the channel bottom will take place as long as that discharge or sequence of discharges and corresponding sediment supply remains the same.

The channel design procedures, therefore, required the development and application of functional relations correlating the geometry, slope, and roughness of stream channels; the grain-size composition of the bed; water discharge; sediment transport capacity; and sediment supply. Inasmuch as the silt clay fractions are not contained in the streambed in appreciable quantities and as these sizes can be expected to pass on through the open river channel and small navigation pools as wash load without appreciable deposition, the design procedures considered only the bed-material load or sand fraction. The relations and their application with particular reference to the channel reach between Arkansas Post and Dardanelle Dam are described.

In brief, the procedure involved the selection of the critical sediment supply that the design channel must transport with each of several water discharges; the trial assumption of a contracted channel width; the determination of the depth and slope required to transport the given discharge and sediment supply; and the corresponding channel bottom grade elevation. If the resulting channel bottom elevation did not provide the required navigable depth below the flat pool level, a new channel width was assumed and the process was repeated until the navigable depth was attained. If the required contracted channel width was less than approximately 1,000 feet, the pool elevation and dam site were revised as necessary.

Other major factors governing the pool elevations and locations for the dams were the effects of the pools on surface drainage and ground water levels on adjacent lands, the effect of channel alinement on navigation conditions in the approaches to the locks, availability of sufficient channel width to provide space for the lock and an adequate spillway, and, to a lesser extent, the foundation and abutment conditions. Provision of adequate harbor facilities was a factor at some localities.

Natural Sediment Load

A rating curve of suspended bed-material load of various sand sizes versus discharge at Little Rock for natural conditions is shown on figure 2. This curve was based on numerous measurements over a period of several years at a bridge at Little Rock. This curve was used to determine the sediment supply in the part of the river downstream from Little Rock to which it was assumed that load reduction effects of the reservoirs would not have progressed by the time of start of navigation.

Development of Relation Between Channel Depth and Slope and Sediment Load

For flexibility in the channel design studies, it was considered desirable to express the relation between stream and channel characteristics, discharge, and bed-material sediment load in generalized terms, so that the effects of

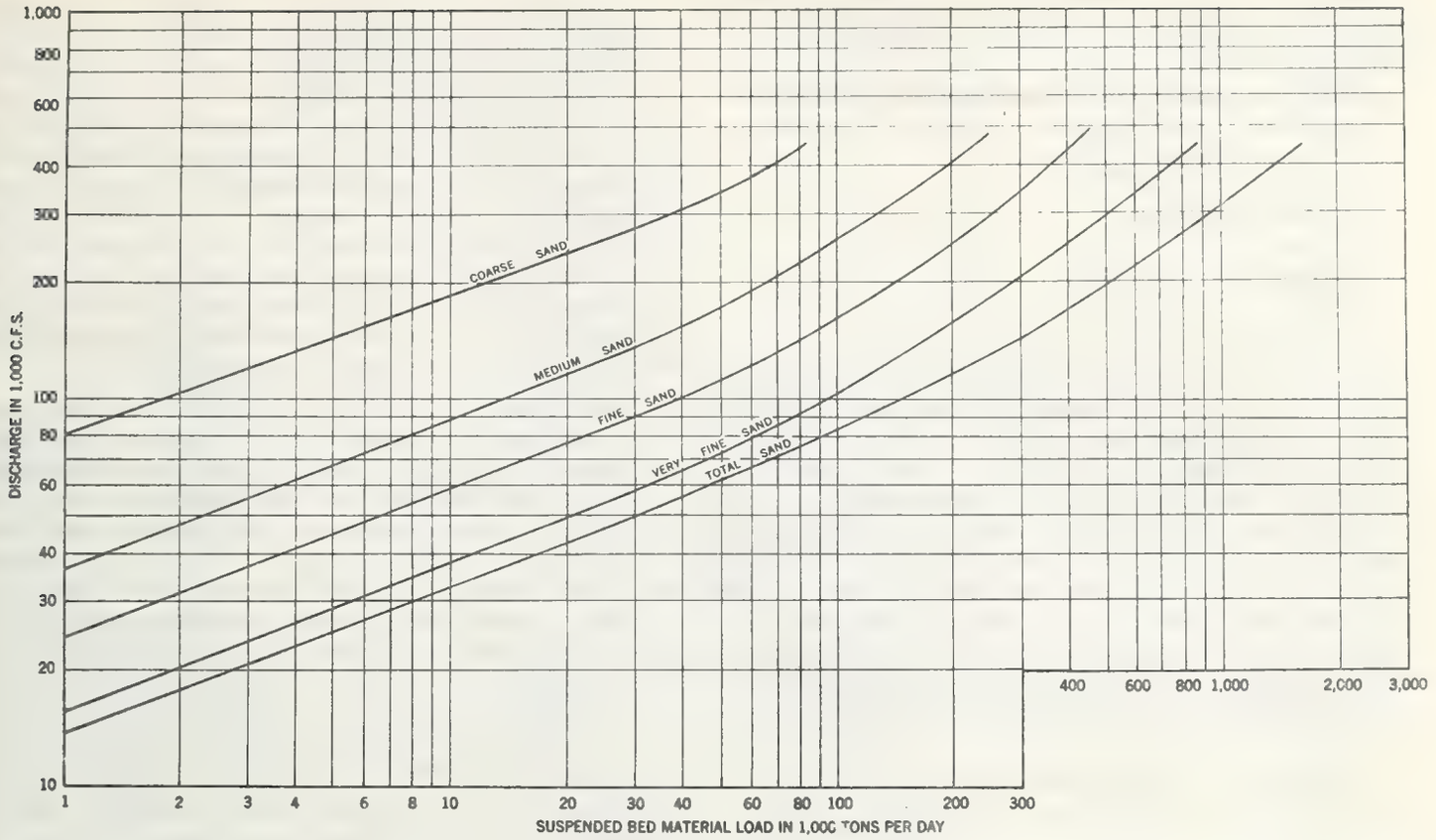

FIGURE 2. — Rating curve of suspended bed-material sediment load versus discharge for the Arkansas River at Little Rock, Ark.

changes in the various parameters involved could be valuated. A functional relation developed empirically by Emmett M. Laursen [1] was used as the framework for developing a generalized working curve for use in the Arkansas River channel studies. Laursen's relationship was adopted for use, because it is expressed in terms that permit separating readily the effects of the various parameters which are generally considered to govern the relation between the bed-material sediment load, the hydraulic characteristics of the streamflow, and the characteristics of the bed material. In addition, being empirical, the Laursen relationship is susceptible of being adjusted to fit Arkansas River observations in a manner similar to the adjustment to fit Missouri River data described by D. C. Bondurant.[2] An average relationship curve, based on data collected in three sets of detailed sediment load and velocity measurements made on the Arkansas River in the vicinity of Dardanelle and Morrilton and plotted in the form used by Laursen, is shown on figure 3 together with the curve from Laursen's paper. As the Arkansas River sediment load measurements included the suspended load only, the plotted data have been adjusted to approximate the total bed material load by adding in values read from the curve designated "bedload" on Laursen's graph. The bedload, or load moving in contact with the bed, represents only a very small part of the total bed material load. The curve based on Arkansas River data falls in the general range of data presented by Mr. Bondurant for the Missouri River at Omaha. The curves in figure 3 are plotted as a relation between the expressions $\frac{\sqrt{\tau_o/\rho}}{w}$ and $\frac{c}{\left(\frac{d}{D}\right)^{7/6}\left(\frac{\tau_o'-1}{\tau_c}\right)}$, where $\sqrt{\tau_o/\rho}$ is the shear velocity at the streambed in feet per second, τ_o is the boundary shear or tractive force at the streambed in pounds per square feet, τ_o' is the boundary shear associated with the sediment particles in the streambed, τ_c is the critical tractive force for beginning of movement of the sediment particles, ρ is the mass density of fluid (1.94 for water), w is the fall velocity of sediment in feet per second, c is the concentration of sediment in percent by weight, d is the diameter of sediment

[1] LAURSEN, E. M. THE TOTAL SEDIMENT LOAD OF STREAMS. Amer. Soc. Civil Engin. Proc. 84, HY1 (Paper 1530). 1958.

[2] BONDURANT, D. C. DISCUSSION OF REFERENCE LISTED IN FOOTNOTE 1. Amer. Soc. Civil Engin. Proc. 84, HY6: 64-74.

particle (mean diameter of fractional size range) in feet, and D is the depth of flow in feet.

Additional pertinent equations are as follows:

$$\tau_o = \gamma DS = 28.25 \frac{n^2 V^2}{D^{1/3}}$$

$$\tau_o' = \frac{V}{30}\left(\frac{d_m}{D}\right)^{1/3}$$

$$\tau_c = 4d$$

$$q_s = 27\, qc$$

where γ is the specific weight of water (62.4 lb./cu. ft.), S is the energy gradient in feet/feet, n is the Manning roughness coefficient, V is the mean velocity (q/D), d_m is the median size of sediment particles in streambed in feet (considered representative of the grain roughness of the bed), q is the discharge per foot width in cubic feet per second per foot, and q_s is the sediment load in tons per day per foot width. Data on grain sizes and corresponding fall velocities for the various size classes considered in this study are given in table 1.

TABLE 1.—*Grain sizes and fall velocities*

Grain-size class	Range	Geometric mean		Fall velocity (w) at 65° F.
	Mm.	*Mm.*	*Ft.*	*Ft./sec.*
Very fine sand	0.062–0.125	0.088	0.000285	0.021
Fine sand	.125– .250	.1767	.000580	.068
Medium sand	.250– .500	.3533	.001158	.15
Coarse sand	.500– 1.00	.707	.00232	.26

For use in the channel design studies discussed in this paper, the Laursen sediment load relationship as modified by the Arkansas River data was converted into a more directly usable general relationship between bed material sediment load per foot width (q_s) and the boundary shear (τ_o) or equally applicable product of depth and slope (DS). This relationship is shown on figure 4,*B*, a separate curve being given for each grain-size class range from very fine sand to coarse sand, inclusive. This relationship was developed for each grain-size class if various combinations of depth and velocity are assumed; corresponding values of $(\frac{d}{D})^{7/6}$, $\left(\frac{\tau_o'}{\tau_c}-1\right)$, $\frac{\sqrt{\tau_o/\rho}}{w}$, and τ_o or DS are computed; the corresponding value of c is obtained from the modified Laursen relationship on figure 3; and the corresponding value of q_s is computed. The values of $\sqrt{\tau_o/\rho}$ and τ_o or DS were computed from the assumed depths and velocities by means of the equation given above when τ_o is expressed in terms of Manning's flow formula. With a constant Manning's n, the curves of load versus DS or τ_o shown on figure 4,*B* for each grain size were uniquely defined for all combinations of depth and velocity or equivalent combinations of depth and slope. The sediment load versus DS curves were computed initially by the use of an arbitrarily assumed Manning's n value of 0.023 in the determination of τ_o and DS. Similar sets of curves were computed for various other n values. By comparison of the various sets of curves, a relationship giving the relative variation of load with variation in Manning's n was developed and is shown as a solid line in the graph on figure 4,*A*.

Subsequently, it was found that for the channel studies discussed in this paper the load curves of figure 4,*A* would correlate somewhat better with backwater computations and data from the total discharge versus total bed material load relation of figure 2 if a basic n value of 0.028 were assumed. The value of 0.028 also agreed more closely with n values determined for the Arkansas River data used in modifying the Laursen relationship. An adjusted curve of relation between relative load and n values for a basic n of 0.028 is shown as a dashed line in figure 4,*A*. The curve agrees closely with independently determined load correction factors obtained by correlating average DS and n size for the Arkansas River downstream from values from figure 2. The independently determined load correction factors are plotted for comparison as circled points on figure 4,*A*. In effect, the correction factors serve to establish the position of curves of load per foot width versus DS for any n value parallel to the basic curves for each grain size.

The curves for the various grain-size classes shown on figure 4,*B* were computed in such a way that each curve gives the load per foot width that would result for a streambed composed entirely of each of the individual size classes. In determining the load for a bed composed of a sand mixture, the load value for each size class is weighted by the percentage factor by which that size class is represented in the bed mixture, and the weighted load values for the various sizes are then summed to obtain the total bed material load. A bed material size (d_m) of 0.25 mm. was used as the representative size for grain roughness in the bed in computing the curves of figure 4,*B*. This size was approximately the average median bed material values from backwater computations with load Dardanelle Dam under present conditions. Other sets of curves based on other median bed material sizes can be developed for general use at other locations having different bed composition and for use in such studies as degradation studies, which require consideration of varying bed grain roughness and varying bed

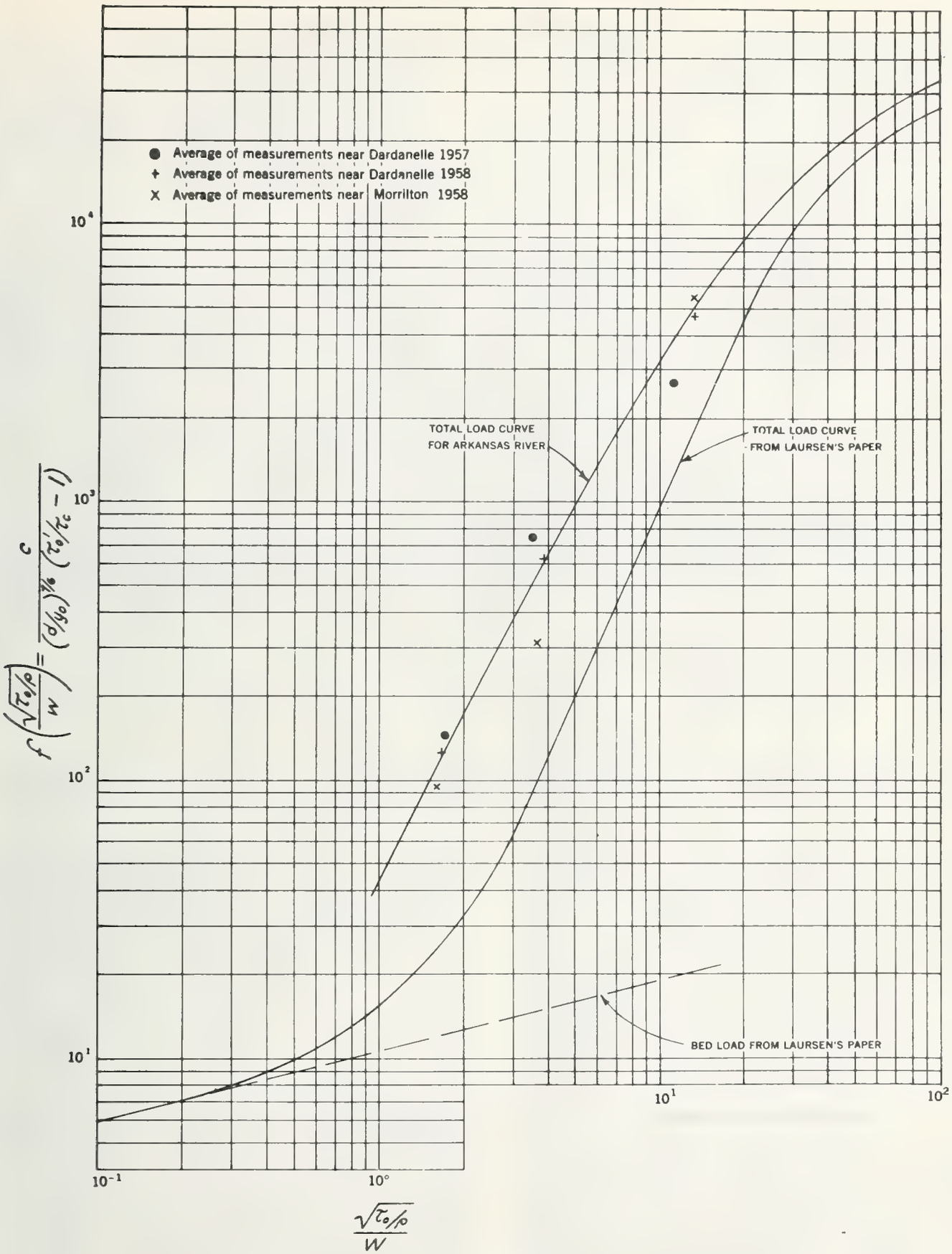

FIGURE 3. — Laursen sediment load function.

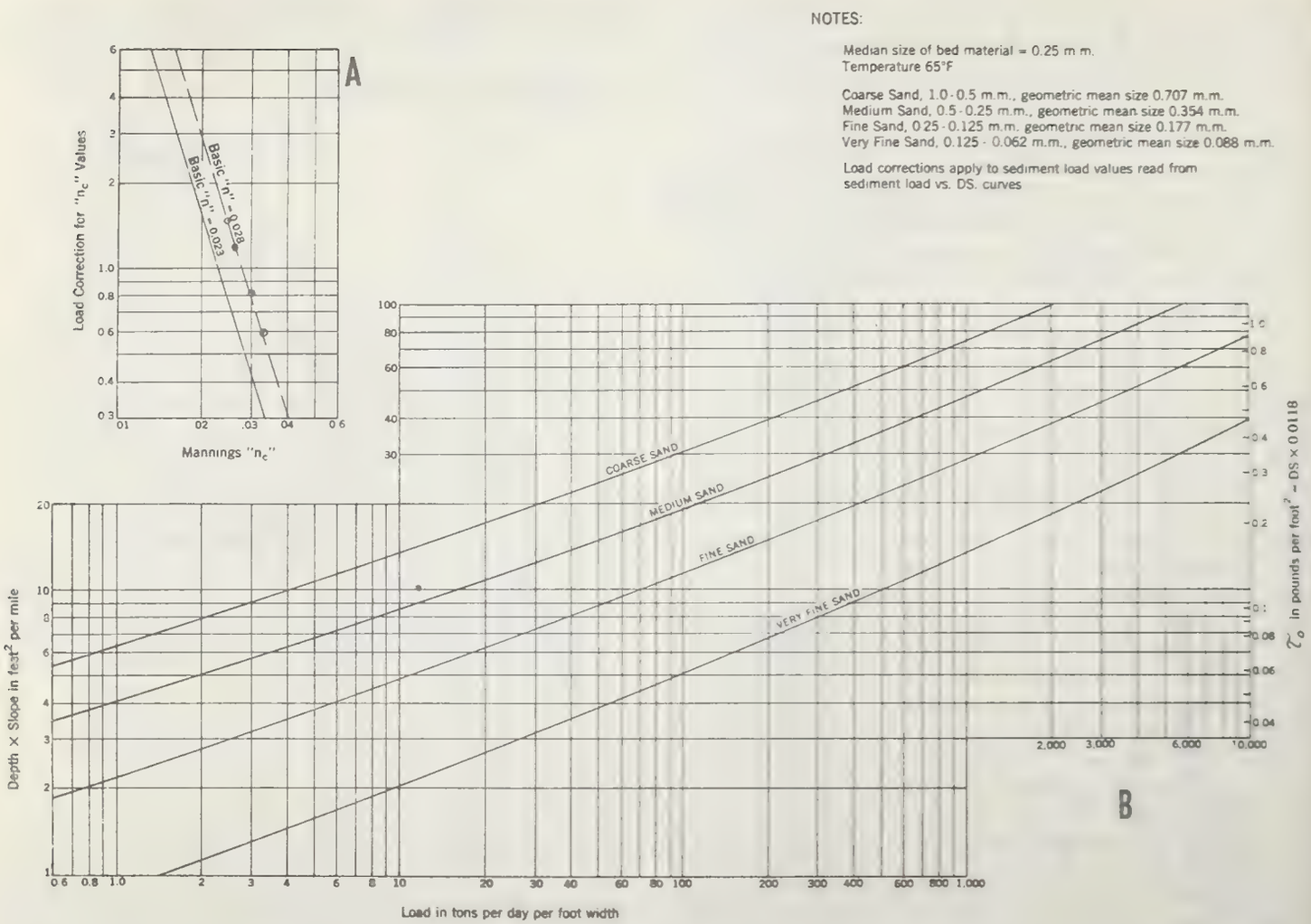
NOTES:
Median size of bed material = 0.25 m.m.
Temperature 65°F
Coarse Sand, 1.0 - 0.5 m.m., geometric mean size 0.707 m.m.
Medium Sand, 0.5 - 0.25 m.m., geometric mean size 0.354 m.m.
Fine Sand, 0.25 - 0.125 m.m. geometric mean size 0.177 m.m.
Very Fine Sand, 0.125 - 0.062 m.m., geometric mean size 0.088 m.m.
Load corrections apply to sediment load values read from sediment load vs. DS. curves
A
Load Correction for "n_c" Values
Basic "n" = 0.028
Basic "n" = 0.023
Mannings "n_c"
B
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
Depth × Slope in feet² per mile
τ_o in pounds per foot² = DS × 0.0118
Load in tons per day per foot width

FIGURE 4. — *A*, Load correction factors for variations in Mannings "n" (main river channel); *B*, relation between bed tractive force, τ_o, or product of depth and slope, *DS*, and bed-material sediment load per foot width.

grain size distributions. However, in the computation of the equilibrium bed profile for dredging in the studies covered by this paper, the bed material has been assumed to be constant at the present average composition. In addition, it was found that with the present average bed composition, the fine sand load curve is approximately representative of the total bed material load. The fine sand load curve of figure 4,*B* has, therefore, been adopted as the basic working curve for these studies.

Modified Bed Material Load

The modified sediment outflow from Dardanelle was estimated for a 50-year period as mentioned previously. The computed bed material load outflow ranges from 2.4 million tons per year at start of operation to 1 million tons per year about 12 years after all upstream reservoirs have been in operation. Thereafter it increased again to about 2 million tons per year as the storage capacity in the upstream reservoirs are depleted after 50 years. From a practical standpoint, these changes in sediment outflow during the 50-year period are negligible. Although a greater percentage of the sand load would pass through the spillway as the reservoir capacity was depleted, the inflowing sand to Ozark and Dardanelle Reservoirs will be reduced in a compensating manner with coarsening of the upstream riverbed, which will be the main source of supply. The sediment load outflow for any discharge was available during the 50-year period for use in the channel studies.

In separate studies for estimating the amount and time sequence of degradation downstream from Dardanelle Dam, the increase in bed material load downstream from Dardanelle was estimated by reaches for a distance of about 40 miles, or about halfway to Little Rock, during the 25-year period after impoundment of the reservoir. Of particular interest herein is the modified bed material load at the time of start of navigation. The minimum depths for navigation should prevail at that time because, with the degrading tendency under reduced sediment

load, the bed level should be progressively lowered thereafter with the passage of time. For the purpose of studying the navigation channel, the sediment load 6 years after closure of Dardanelle Dam has been assumed to conform to start of navigation. The total bed material load plotted versus distance downstream from Dardanelle in the sixth year is shown in figure 5 for two discharges for purpose of illustration. The load of sand progressively increases downstream to the vicinity of Little Rock, where it is estimated that essentially complete recovery of the original or natural load will have occurred in the sixth year. This means that at the time, degradation of the channel will be in progress between Dardanelle and Little Rock, but little significant degrading could be occurring downstream from there, since the river would be carrying a capacity load of bed material. The use of the natural and modified sediment load curves in estimating the future channel is described subsequently.

Manning's n Values

The channel studies required separation of the channel and overbank discharges both for purpose of making backwater computations and in estimating the movement of the sediment load. Highwater marks and crest stages have been obtained at various times in the past, which, in conjunction with stages at gaging stations along the river, permitted drawing of water surface profiles for a range in discharges. When backwater for rises were computed near bankfull stages, main channel (n_c) values were determined, and by use of those values the overbank discharges could be computed for rises exceeding bankfull. Work of this nature has been done at various times since 1943. These data were reviewed and additional estimates of main channel n values were made by means of trial backwater computations to check observed profiles based on more recent floods. As few high-water soundings are available, the cross section elements used to compute the n values were based largely on low-water surveys. The channel (n_c) values adopted increased as the discharge decreased from 0.025 for 435,000 c.f.s. to 0.033 at 50,000 c.f.s., as shown on figure 6. Manning's n values determined for use in the studies described herein are somewhat higher than those customarily used for streams of this type, owing to the fact that weighted average or

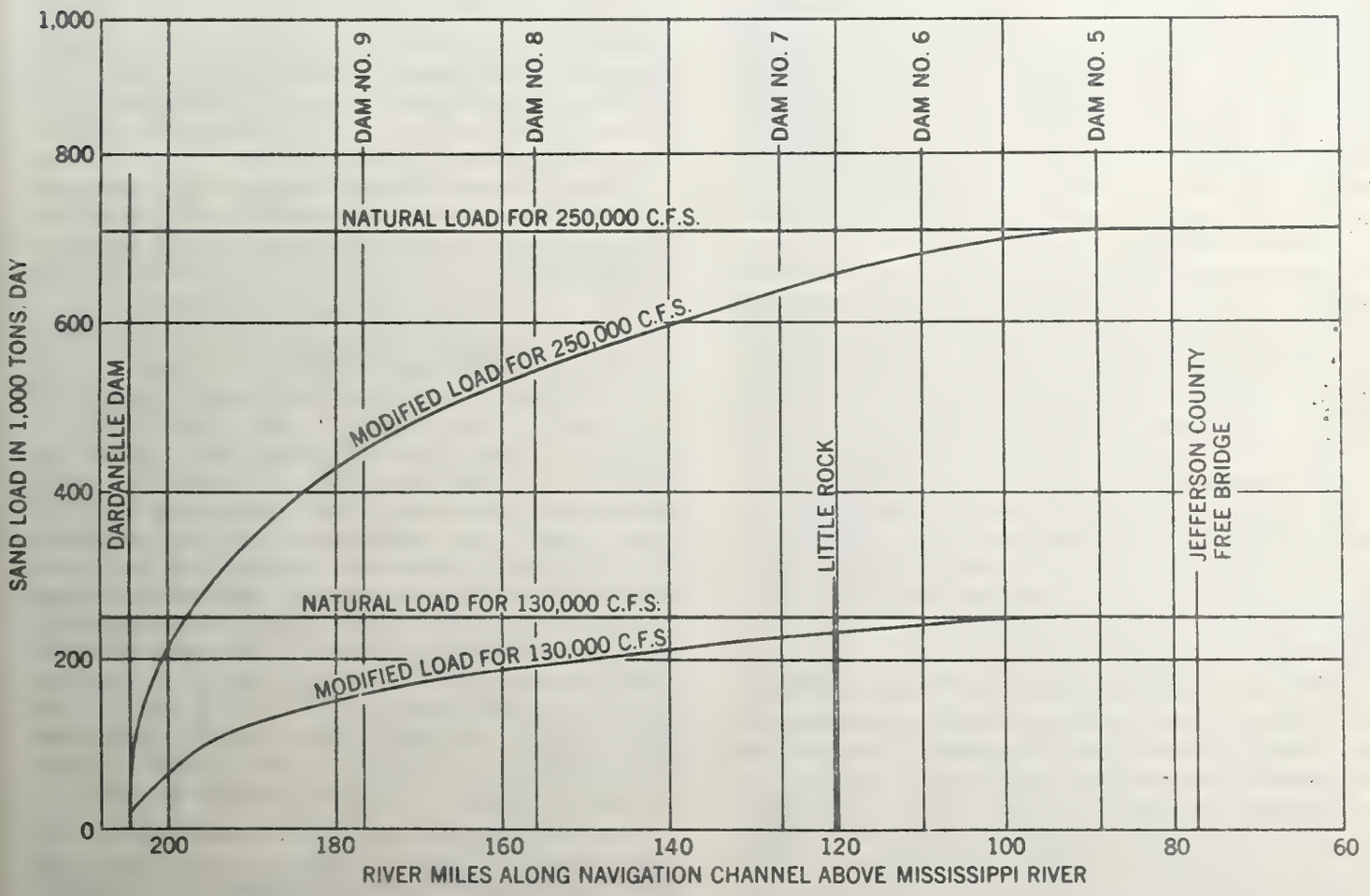

FIGURE 5. — Estimated variation of modified sand load with distance downstream from Dardanelle Dam.

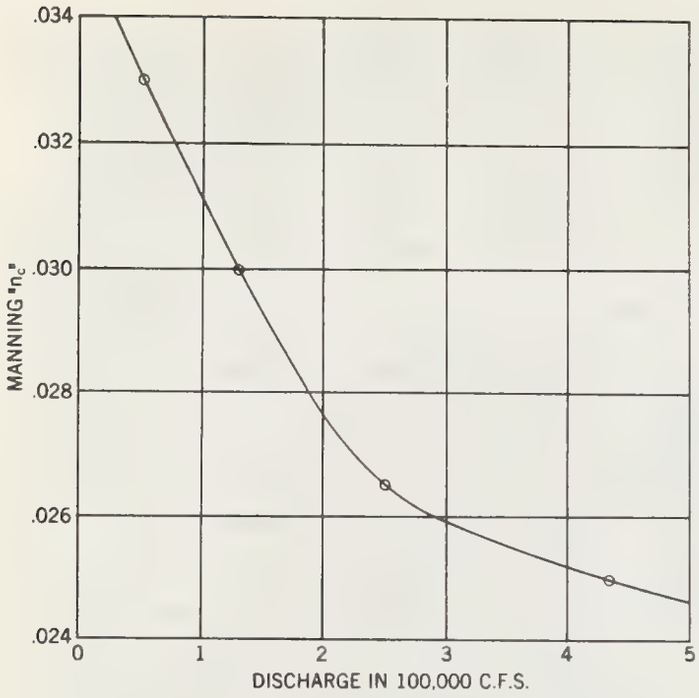

FIGURE 6. — Variation of the main channel Mannings "n" values with total discharge on the Arkansas River in the vicinity of Little Rock.

effective depths and corresponding effective widths have been used in these studies as discussed later. The corresponding cross section conveyance factor $\Sigma\ (AD^{2/3})$ is somewhat larger than the conventional $AD^{2/3}$ values computed from the total area and mean depth or hydraulic radius. Higher n values are naturally associated with larger conveyance factors in Manning's formula.

The overbank was divided into areas that differed in ground cover or land use, and overbank (n_o) values were assigned for these areas by a study of the latest aerial mosaics and published data on roughness coefficients for flood plain areas.[3] [4] [5] Typical n_o values were in the range of 0.04 for sandbars with scattered low brush, 0.06 for pastureland with low grass and scattered timber, and 0.13 for moderately thick willow growth. A high degree of accuracy in the overbank (n_o) values was not essential since the percentage of total flow carried by the overbank is small. For example, under present conditions between Little Rock and Pine Bluff, 93 percent of a total flow of 250,000 c.f.s. and 89 percent of a total flow of 435,000 c.f.s. are carried by the main channel.

[3] BRADLEY, J. N. HYDRAULICS OF BRIDGE WATERWAYS. U.S. Dept. Commerce, Bur. Pub. Roads, Hydraul. Design Ser. 1, 34 pp. 1960.

[4] RAMSER, C. E., and BARTZ, H. J. THE FLOW OF WATER IN THE MAIN DIVERSION FLOODWAY OF THE LITTLE RIVER DRAINAGE DISTRICT IN SOUTHEAST MISSOURI. U.S. Dept. Agr. Bur. Pub. Roads, Mimeo. Paper. 1924.

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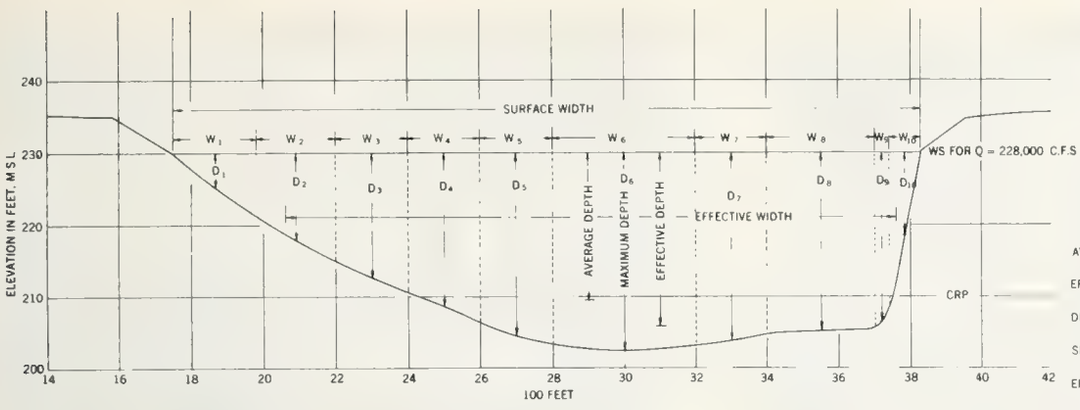
Adjustment of Cross Sections to Stabilized and Realined River Channel Conditions

The ultimate objective of the Arkansas River channel studies was to estimate the cross section elements of the river at the time of start of navigation with the channel realinement of all dams and stabilization works in place. However, it was necessary to proceed a step at a time, starting with the available cross sections of the river. The first step was to adjust tentatively available cross sections of the existing channel for realinement of the river with the normal bank stabilization works in place. The 186-mile reach of the Arkansas River between Dardanelle and Arkansas Post was divided into about 200 subreaches approximately one-half to 2 miles in length, and a typical cross section was developed for each subreach and the conveythese cross sections were of three general types: Crossing sections, sections representative of the upstream part of a bend, and sections representative of the downstream part of a bend. Where recent surveyed ranges were available, an average section for surveyed conditions was developed for each subreach and the conveyance factors of the cross section for a range of flows was computed by $\Sigma\ (AD^{2/3})$ where A is the area and D is the average depth in subsections of the cross section. Figure 7 illustrates how a typical cross section was divided into subsections at the major breakpoints.

These average cross sections for surveyed conditions were then modified to reflect the changes that could be expected as a result of stabilization and rectification work fixing the banks, modifying the radius of curvature of bends, contracting the channel to concentrate the flow in wide and shallow, or braided, sections into a single deep channel, and modifying the slope of the river by the construction of cutoffs. These modifications were based on studies of characteristics of existing stable reaches of the stream, their shape, width, maximum depth, and sediment carrying capacity. Design curves used as a guide for maximum depths in bends as a function of radius of curvature and depths of crossings as a function of width are shown on figure 8. The plane of reference for the depths on this figure is the construction reference plane (C.R.P.) used in the construction of bank stabilization structures. The C.R.P. corresponds to the water surface profile for a flow of approximately 10,000 c.f.s.

As a first approximation it was assumed that the subreaches would have approximately the same conveyance factor for various discharges after stabilization and rectification as at the



SUB SECTION	WIDTH	DEPTH	AREA	$D^{2/3}$	$AD^{2/3}$	$DAD^{2/3}$
1	230	4.9	1,130	2.88	3,250	15,900
2	220	12.2	2,680	5.30	14,200	173,200
3	200	17.4	3,480	6.72	23,390	407,000
4	200	21.3	4,260	7.68	32,720	696,900
5	200	25.4	5,080	8.64	43,890	1,114,800
6	400	27.6	11,040	9.13	100,800	2,782,100
7	200	26.3	5,260	8.84	46,500	1,223,000
8	300	24.9	7,470	8.53	63,720	1,586,600
9	40	23.6	940	8.23	7,740	182,700
10	90	11.5	1,040	5.10	5,300	61,000
TOTAL	2080		42,380		341,510	8,243,200

$$\text{AVERAGE DEPTH} = \frac{\text{TOTAL AREA}}{\text{SURFACE WIDTH}} = \frac{42,380}{2,080} = 20.4 \text{ FT}$$

$$\text{EFFECTIVE DEPTH} = \frac{\text{TOTAL } DAD^{2/3}}{\text{TOTAL } AD^{2/3}} = \frac{8,243,200}{341,510} = 24.1 \text{ FT}$$

$$\text{DEPTH RATIO} = \frac{\text{AVERAGE DEPTH}}{\text{EFFECTIVE DEPTH}} = \frac{20.4}{24.1} = .846$$

$$\text{SHAPE FACTOR} = \frac{\text{AVERAGE DEPTH}}{\text{MAXIMUM DEPTH}} = \frac{20.4}{27.6} = .739$$

$$\text{EFFECTIVE WIDTH} = \frac{\text{TOTAL } AD^{2/3}}{\text{EFFECTIVE } D^{2/3}} = \frac{341,510}{24.1^{2/3}} = \frac{341,510}{201.1} = 1,700 \text{ FT}$$

$$\text{WIDTH RATIO} = \frac{\text{SURFACE WIDTH}}{\text{EFFECTIVE WIDTH}} = \frac{2,080}{1,700} = 1.224$$

FIGURE 7. — Subdivision of a main channel cross section and determination of the "effective mean depth" and "effective width."

present time; therefore, the value of $\Sigma\ (AD^{2/3})$ was kept approximately the same when the cross section was adjusted for improved channel conditions. For example, where the concave bank in a bend was stabilized and an appreciable change was made in the radius of curvature of the stream, the cross section was modified to conform generally to an existing stable section of comparable radius of curvature and the depths were refined so that $\Sigma\ (AD^{2/3})$ remained essentially constant. In subreaches where the channel had changed significantly since the latest survey, hypothetical sections were developed from the latest aerial mosaics, earlier surveys, and the characteristics of similar stable sections.

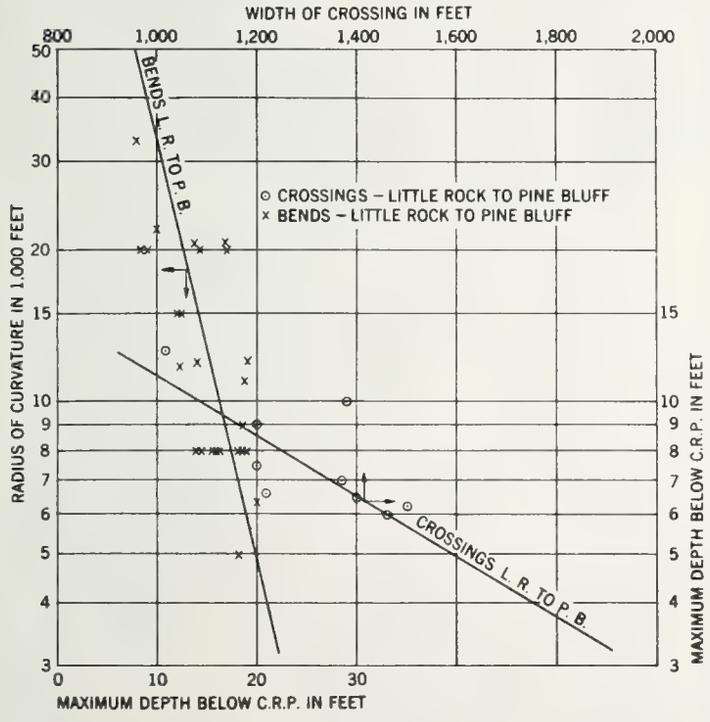

WIDTH OF CROSSING IN FEET
BENDS L. R. TO P. B.
CROSSINGS L. R. TO P. B.
○ CROSSINGS – LITTLE ROCK TO PINE BLUFF
× BENDS – LITTLE ROCK TO PINE BLUFF
RADIUS OF CURVATURE IN 1,000 FEET
MAXIMUM DEPTH BELOW C.R.P. IN FEET
MAXIMUM DEPTH BELOW C.R.P. IN FEET

FIGURE 8. — Relations between radius and curvature and maximum depth in bends and between channel width and maximum depth in crossings.

Backwater Computations

Backwater computations were made for a range of discharges up to the maximum modified discharge of record by the use of the tentative cross sections described above. Separate channel and overbank distances were determined as required for reaches represented by the respective cross sections. The Manning formula is applicable in the form shown below. Plots of water surface elevation versus the applicable ratios in the formula were prepared for convenience in making the computations.

$$\text{Fall} = \left[\frac{\dfrac{Q_t\ L_c^{1/2}}{1.49}}{\dfrac{A_c D_c^{2/3}}{n_c} + \dfrac{A_o D_o^{2/3}}{n_o}\left(\dfrac{L_c}{L_o}\right)^{1/2}} \right]^2$$

Where Q is the discharge, L is the length of reach, A is the cross-sectional area, D is the effective mean depth, and n is the friction factor. The subscript t is total, c is the movable bed part of the cross section, o is the nonmovable bed part.

The primary departure from the usual method of computing backwater is the use of the "effective mean depth" and "effective width." As described subsequently herein, the shape of the cross section of the channel must be defined in terms of the water surface width and mean depth. The channel shape varies widely from near-rectangular on crossings to triangular in sharp bends. Therefore, the depths and widths were weighted according to the concentration of discharge across the section or by the conveyance factor $(AD^{2/3})$ of subareas to obtain effective mean depths and widths. As illustrated in figure 7, each cross section was divided into subsections at major breakpoints in the bottom and the overbank and channel were also separated.

The factor $AD^{2/3}$ was computed for each subsection and multiplied by the depth in the subsection ($D\ AD^{2/3}$). The effective depth is then computed as shown in the tables in figure 7 as:

$$\text{Effective } D = \frac{\Sigma\ (D\ A\ D^{2/3})}{\Sigma\ (A\ D^{2/3})}$$

and the effective width $= \dfrac{\Sigma\ (AD^{2/3})}{D^{5/3}}$

For a perfectly rectangular channel the conventionally determined actual mean depths and actual surface widths are the same as the effective values. The effective mean depth and effective width in effect define an equivalent rectangular section and the nonrectangular sections.

Adjustment of Cross Sections for Continuity of Natural Sediment Load

In an alluvial stream like the Arkansas River, the channel cross section at any location is developed by the flow pattern under the prevailing discharge, the bed material load, and the type of material comprising the bed. Most of the available cross sections were obtained from surveys made at low stages, and the riverbed configuration reflected the accumulated effect of the random flow conditions that existed immediately before the survey was made. Surveys made at the same locations have indicated that the bed elevation may vary as much as 20 feet because of differences in antecedent flow conditions. Also, the riverbed surveyed at a given high stage may range widely, because the discharge and sediment load were not constant for a sufficient period to develop a typical shape for that stage. Therefore, to afford an adequate and systematic basis for study of the channel, it was considered necessary to adjust further the channel depths of the realined channel for uniform flow and constant natural sand load conditions for each of the several discharges ranging up to about the maximum modified flood of record. The method used involved the combination of the basic relation of bed material load per foot width of channel, as described previously, with the Manning flow formula.

The Manning formula was arranged in the following form:

$$D = \left(\frac{5{,}280 \times q_c^2 \times n_c^2}{2.21\ DS}\right)^{3.7}$$

Where D is the effective mean depth of the movable bed part of the main channel, q_c is the discharge per foot of width of main channel, or movable bed channel, S is the hydraulic slope (assumed to be the same or the water surface slope), n_c is the channel roughness factor for the applicable river discharge, and DS is the product of the mean depth and slope. The constant 5,280 is introduced in the equation to permit use of the slope S in feet per mile and DS in feet squared per mile.

An example for adjusting the mean depth at a cross section to equilibrium conditions for a uniform load is shown below.

Given:

Total river discharge, c.f.s.	250,000
Main channel discharge, c.f.s.	241,000
Effective width of main channel, feet	1,385
q_c, c.f.s.	174
Total natural sand load for $n =$ 0.0264 from figure 2, tons/day	710,000
q_s, sand load per foot of main channel width, assuming all sand carried by main channel, tons/day	512
q_s for n_c of $0.028 = 512 \times 0.865 =$ $512 \times 1/1.157$, where the load correction factor 1.157 is obtained from the circled point for $n =$ 0.0264 on figure 4,*A*	443
DS from fine sand curve on figure 4,*B*, sq. ft./mile	20.5

Effective mean depth adjusted,

$$D = \left(\frac{5{,}280 \times (174)^2 \times (0.0264)^2}{2.21 \times 20.5}\right)^{3.7} = 28.4 \text{ feet}$$

The step for computing q_s for $n_c = 0.028$ outlined above is equivalent to and substitutes for making use of a separate load curve for an n value of 0.0264 to determine DS. As discussed previously, the separate load curve, if shown, would be parallel to the basic fine sand curve for $n = 0.028$ on figure 4,*A*, and its position is defined relative to the basic curve by the load correction factor 1.157 for an n value of 0.0264. In effect, the basic curve is used only to establish the slope of the functional relationship between load per foot width and DS.

If the main channel is contracted, as with dikes, the sand load per foot of width is increased, and from figure 4,*B* it results in an increase in DS, which appears in the denominator of the equation. However, the discharge per foot of width of the movable bed (q), which appears in the numerator, will increase more, so that the depth is always increased for a contraction of the channel. Furthermore, a reduction in sand load per foot width results in a reduction in DS (fig. 4,*B*), which also results in an increase in the depth. Attention is invited to the assumption that all the sand load is carried by the main channel. Although the finer silt and clay particles are uniformly dispersed, the heavier sand load is largely concentrated in the deeper channel. As stated previously, about 90 percent of all the flow is carried by the main channel. Computations for a typical cross section indicate that more than

99½ percent of the sand load is carried in the main channel.

The effective mean depth adjusted for uniform flow of water and sand was determined for each cross section used in the backwater computations. However, to compute the bed elevation, an actual rather than effective depth must be subtracted from the water surface. The actual mean channel depth was known for the basic plotted cross sections by dividing the cross-sectional area by the surface width. The effective mean depths were also computed for the basic cross section. The ratio of the actual and effective mean depths determined from the basic cross sections times the *adjusted* effective mean depths gave the adjusted actual mean depths.

Subtraction of the actual mean depth from the water surface profiles obtained by the backwater computations would afford a separate mean bed profile for each of the discharges. However, the bed elevation of the deeper part of the river comprising the navigation channel is needed rather than the mean bed level across the entire main channel. The highest bed elevation in the lowest 250 feet width of channel was adopted for measuring navigable depths, since this is about the minimum width for navigation. As the shape of the cross section depends on the direction of flow and the flow pattern, the depth in the 250-foot width cannot be computed analytically. Therefore, empirical relationships were developed between mean depth and the depth in the deeper 250-foot width, based on the surveyed cross sections, as shown on figure 9. Figure 9, left, shows the mean depth versus the depth for 250 feet with channel width as parameter for a near-rectangular-shaped cross section. The actual adjusted mean depth and the actual adjusted surface width were used to enter the figure. The correction factors for shapes other than rectangular ones were obtained from figure 9, right.

With the final actual depths for the 250-foot width of realined channel under uniform natural sediment load condition obtained, those depths were subtracted from the water surface profiles estimated by backwater computations to obtain the bed elevations. The water surface profiles and corresponding bed elevations for 50,000 and 250,000 c.f.s. are shown for illustration for pool No. 7 on figure 10. The effect of discharge on the bed profile depends on the shape of the cross section. In general, the larger discharge tended to lower the computed bed profile in bendways or where the overbank flow is relatively small and to raise the bed level on crossings, which is in accordance with observations of the Arkansas River and other alluvial streams.

Water Surface Slope

In the above solution no changes or corrections were made in the water surface slopes or elevations from those computed in the backwater computations. However, for continuity of sediment load with the depth (D) computed

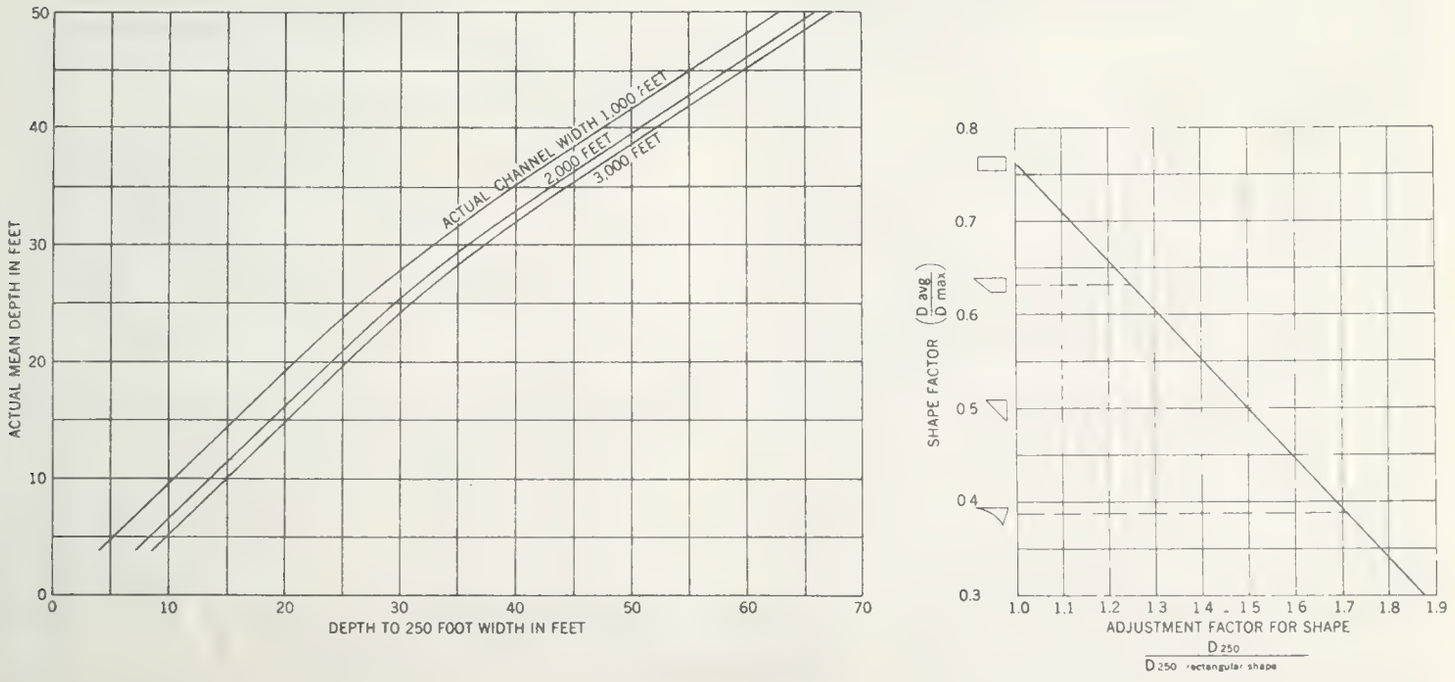

ACTUAL MEAN DEPTH IN FEET
DEPTH TO 250 FOOT WIDTH IN FEET
ACTUAL CHANNEL WIDTH 1,000 FEET
2,000 FEET
3,000 FEET
SHAPE FACTOR (D avg / D max)
ADJUSTMENT FACTOR FOR SHAPE
D250 / D250 rectangular shape

FIGURE 9.—Left, relation between mean channel depth, width, and depth to the highest point in the lowest 250-foot width of near-rectangular cross sections; right, adjustment factors for determining depth to the lowest 250-foot width for cross sections of shapes other than near-rectangular.

and DS known, a new slope can be obtained as:

$$\text{Slope} = \frac{DS}{D}$$

If an increase in depth is determined, the slope will always be reduced, provided the channel friction factor n_c remains unchanged, and a correction in slope is then necessary that will in turn change the water surface elevations from those obtained by the backwater computations. The bed elevations are shown on figure 10. However, the channel n_c values were assumed to increase over those to date, because of the extensive revetment and dike system provided under the normal bank stabilization plan.

Estimates made indicated that an increase in n_c value of about 5 percent would compensate for the computed flattening of the slope and raise the water surface profiles back up to those determined by backwater computations. The channel model study at the Waterways Experiment Station, as subsequently described, indicated an increase in n_c values and an increase in slope for high flows as a result of channel contraction. Therefore, no correction was made in the slopes where contraction works alone were involved in channel deepening as is the case for the data described above. Note that an assumption of flatter slopes and the related lower water surface profiles would have resulted in lower bed elevations than shown, which would not have been conservative with respect to establishing the normal pool elevations for navigation. However, the falls through the reaches were computed by the ratio DS/D for use in a subsequent step.

Adjustment of Cross Section Elements for Modified Sediment Load with Dams in Place

With the channel elements determined for natural load conditions for the improved channel as described above, the next step was to make further channel adjustments to obtain the controlling riverbed profiles under modified sediment load conditions at the time of start of navigation (6 years after closure of Dardanelle) and with the navigation locks and dams in place. As described previously, the load would be reduced downstream to the vicinity of Little Rock. First, locations for the dams were assumed, then the new equilibrium bed elevation was computed in the 250-foot width of channel

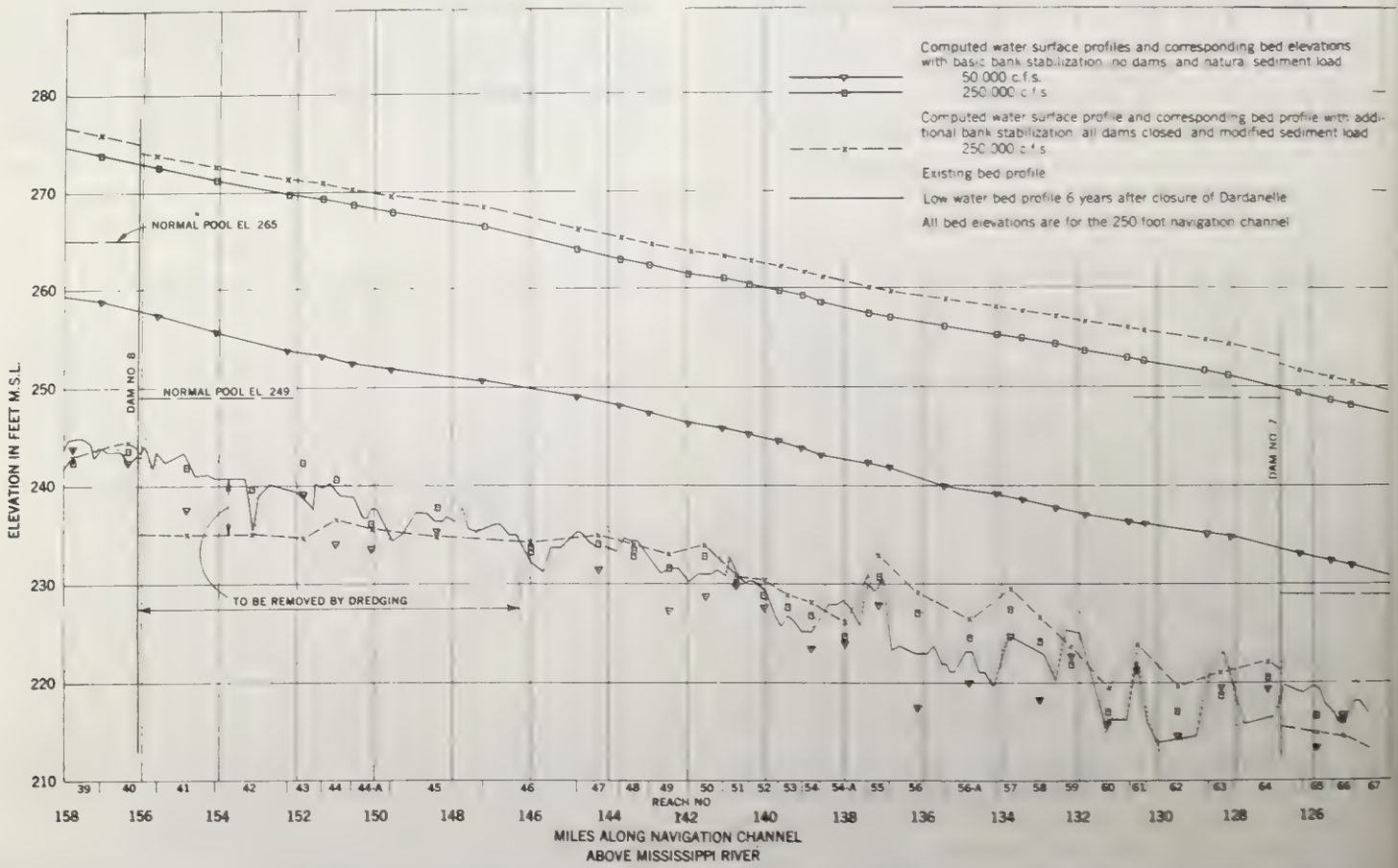

Computed water surface profiles and corresponding bed elevations with basic bank stabilization, no dams, and natural sediment load
50,000 c.f.s.
250,000 c.f.s.
Computed water surface profile and corresponding bed profile with additional bank stabilization, all dams closed, and modified sediment load
250,000 c.f.s.
Existing bed profile
Low water bed profile 6 years after closure of Dardanelle
All bed elevations are for the 250-foot navigation channel
NORMAL POOL EL. 265
NORMAL POOL EL. 249
DAM NO. 8
DAM NO. 7
TO BE REMOVED BY DREDGING
ELEVATION IN FEET M.S.L.
280
270
260
250
240
230
220
210
REACH NO
39 40 41 42 43 44 44-A 45 46 47 48 49 50 51 52 53 54 54-A 55 56 56-A 57 58 59 60 61 62 63 64 65 66 67
158 156 154 152 150 148 146 144 142 140 138 136 134 132 130 128 126
MILES ALONG NAVIGATION CHANNEL
ABOVE MISSISSIPPI RIVER

FIGURE 10. — Streambed and water surface profiles in Arkansas River Pool No. 7 for basic conditions and computed modified regime conditions.

under the modified sediment loads at time of start of navigation. The basic method used was the same as described above. A variation from the previous computations was the adjustment of the fall through each reach for the flattening of the water surface slope, owing to reduced sediment load. This was done by computing the fall through each subreach by the equation $S = \frac{DS}{D}$, using the DS value corresponding to the reduced load, subtracting this value from the fall computed from the same formula as described in the preceding paragraph, and subtracting this difference from the fall determined by backwater computations for the basic cross sections. This gave the adjusted fall in each reach for the reduced sediment load and the accumulation of falls gave the adjusted water surface profile. With the new water surface known and the depth computed, the bottom elevation under modified sediment load for 250,000 c.f.s. was computed.

While the spillway gates would be fully open for the discharge of 250,000 c.f.s., the constriction caused by the piers, lock, and embankments would create a swellhead at each dam. The higher elevations as indicated on figure 10 with the dams in place are caused by this swellhead.

Adjustment of Cross Section Elements for Special Contraction Dikes

As mentioned previously, in addition to the regular bank stabilization and realinement works, special contraction dikes are planned for several miles downstream from each dam to increase the concentration of flow and cause deepening of the channel. The special dikes are similar to the regular dikes except that on crossings a trace width as small as 1,000 feet was provided, as compared to about 1,400 feet for the regular training dikes.

The last step in the computations was to determine the depths and riverbed elevations for the additional contraction of the main channel. With the reduced widths selected, the resulting increased depths and lower bed elevations were computed by the procedure used before. As before, no flattening of the slope was used in those computations where channel deepening is caused by contraction alone. The depths were subtracted from the water surface determined previously for the reduced sediment load conditions. It is apparent that the effect of the reduced sediment load on channel depths and water surface slopes had to be computed before the effect of the additional channel contraction could be determined, rather than combining the computations into a single step. The final equilibrium bed elevation is shown as a dashed line on figure 10.

As previously indicated, the equilibrium bed is the nonscouring or filling configuration created by a certain flow pattern or direction of flow under a specified discharge, load, and bed material size. As the equilibrium bed varies with discharge, computations were required for several discharges to determine which created the highest or controlling bed elevation at the respective cross sections. The equilibrium bed also depends on the bed material sizes, whereas the computations do not provide for any change in the material forming the bed. Any natural degradation would tend to coarsen the bed material. However, since it is planned to dredge in the head of each pool, thus removing part of the bed surface, the bed material at the new bed elevation under equilibrium status would not depart greatly from that for the original cross sections. The effect of the coarsening of the bed where no dredging was performed was taken into account in computing the reduced bed material load, as previously described.

Dredging

Dredging in the head of pools where depths would be critical for navigation is required to attain the equilibrium bed at time of start of navigation, because of the slow natural degradation process and also as armouring by the coarser material would theoretically prevent ever fully reaching the prescribed depth by natural scour. A dredge can operate more efficiently by excavating a deeper and narrower cut than would normally conform to the relatively shallow depth of material above the equilibrium bed level. Therefore, the proposed dredging would remove in a relatively narrow, deep cut, a volume of material equivalent to that between the cross sections of the bed at time of start of navigation and the equilibrium bed cross section, so that the river could readily mold it to the estimated equilibrium shape. A typical dredge cut is indicated on the cross section on figure 11. The bed profile at time of start of navigation without dredging and the final equilibrium bed profile are shown on figure 10, the difference in the profiles indicating the material to be dredged as noted. Ample space for spoiling of the dredged material is available in dike systems and on the relatively high convex bars within the river channel. Placement of the spoil in this manner will provide further contraction to the main channel flow and thus further encourage deepening of the channel by scour of the bed. Since dredging would be undertaken during low water the bed profile at time of start of navigation is shown for a small discharge or low water, and since the crossings that are the shallowest part of the river would tend to be highest during high discharges, the equilibrium bed profile is com-

puted for a high discharge. For purposes of illustration, the equilibrium bed is shown for the discharge of 250,000 c.f.s.

Regulation of the Pools

Another factor related to computing the riverbed level is the plan of regulation of the pools and the backwater created by the locks and dams. No detailed plan of regulation has been studied for each of the pools, but most of the bed material load will be carried by the higher flows when all spillway gates will be fully open, resulting in essentially open river conditions. However, the locks and dams will cause some backwater even with all gates open and a higher computed water surface will tend to raise the bed level correspondingly, since the computed depths are subtracted from the water surface to obtain the bed elevations. In other words, backwater would eventually cause sufficient deposition to attain a higher equilibrium bed than otherwise and the mechanics of the method followed simulated that situation. The effect of backwater at the dams is shown in the example on figure 10.

When pooled stages prevail in conjunction with the smaller discharges there will be a tendency for deposition to occur, but not only is the natural bed material carried at such times small, but the supply from upstream will be cut off at Dardanelle and the relatively slackwater condition in the navigation pools will reduce the tractive force on the bed so that only a relatively small load can be initially entrained from the riverbed to create a deposition problem farther downstream. Therefore, the smaller flows under pooled conditions are expected to have little influence in forming the riverbed.

Channel Model Tests at Waterways Experiment Station

A generalized model study was conducted at the Waterways Experiment Station of a 10-mile reach of the Arkansas River between Little Rock and Pine Bluff to investigate the effects of various types of control works on channel stability and sediment transport with normal sediment load. The model was constructed to a horizontal scale of 1:150 and a vertical scale of 1:36, resulting in a geometric distortion of 4.17. To provide satisfactory sediment transport in the model, a supplementary slope of 0.27 ft. per mile was used, giving a slope scale of 5.7:1. The overbank areas were molded in concrete, and the movable bed portion was originally molded to October 1957 prototype conditions, using sand with a median diameter of 0.2 mm. The time scale was such that a prototype day was reproduced in 10 minutes in the model; the rate of introducing bed material and the discharge scale were varied with stage.

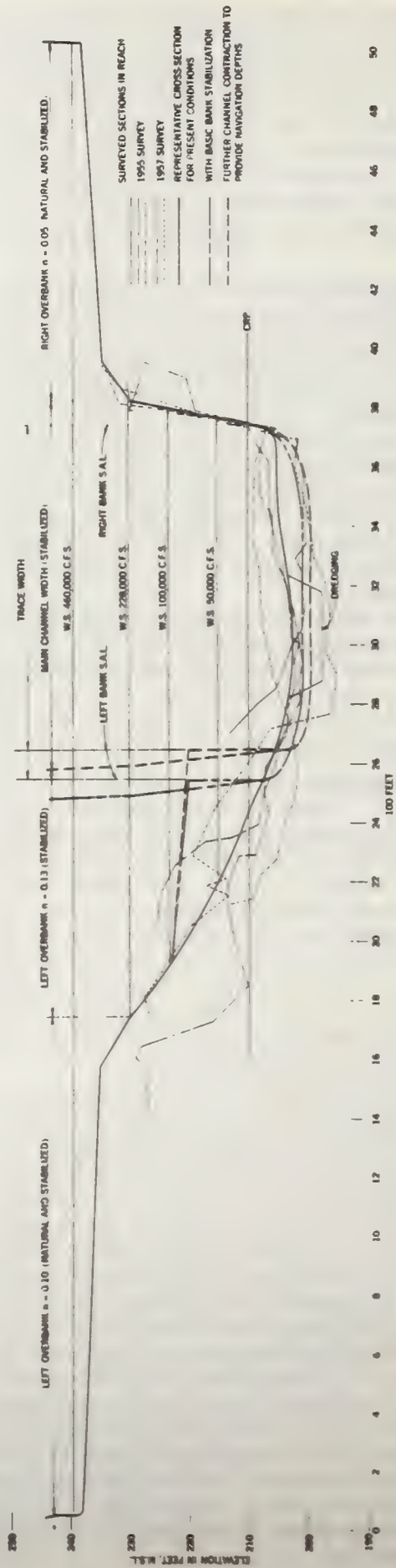


FIGURE 11. — Typical cross section illustrating comparison between present conditions and conditions with channel contraction.

After adjustment of the model, a base test was made with the channel rectified and stabilized according to the normal bank stabilization plan for the river; that is, the concave banks were revetted and the crossings contracted to 1,200 or 1,400 feet. The efficacy of plans of supplemental improvement were then compared to conditions in this base test. Several runs were made for each plan in an attempt to obtain equilibrium sediment load conditions, each run being one reproduction of the experienced 1957 discharge hydrograph.

Because of time limitations most plans were not tested to stability, and the effects of the various features tested were not fully developed. Certain arbitrary assumptions were also made, such as the use of a specific discharge hydrograph for testing and the lower tailwater control to reflect the effect of a dredged channel used in some runs. However the results do indicate that by proper local contraction additional depths can be attained, and the tests provide general indications of the type of effects produced by the various regulative measures investigated as follows:

(1) Installation of a large dredge cut without an appreciable reduction in sediment load and construction of control structures would not be effective, as the dredged cut would soon refill.

(2) Regulating structures, which increase the radius of curvature of a bend, provide a more uniform channel cross section and improve the alinement of the channel over the crossings.

(3) In bends, longitudinal dikes provide a more efficient channel than spur dikes, improve flow conditions and channel cross section, and reduce the amount and extent of shoaling in the crossings.

(4) Extending training works on a concave bank downstream into the crossing improves the alinement of the channel through the crossing, reduces shoaling in the crossing, and improves flow conditions in the next bend downstream by reducing the tendency for a point bar to develop there.

(5) Dredged spoil and willow growth on convex bars improve the alinement, depth, and shape of channel through the bend. Such spoil banks require only minor protection except at the upstream end.

(6) Contracting the channel to 1,000 feet through the crossings increases navigation depths and improves channel alinement. This degree of contraction and the placing of dredged spoil and willow growth on convex bars simultaneously increased water surface slopes at high flows significantly.

(7) Backwater computation for model test conditions indicated that channel n values were greater in reaches with spur dikes than in uncontrolled or revetted reaches, but that with longitudinal structures along the channelward ends of spur dikes the effect of the dikes on the channel roughness would be minimized. Backwater computations indicated that constricting the crossings to 1,000 feet with channel-line structures in bends and simulating accretion and willow growth on convex bars increased n values to approximately those which obtained for spur dikes. Low-level longitudinal groins did not produce any discernible increase in n value.

(8) Spur levees were tested along with other factors, so their effect was not isolated. However, they would be effective only during infrequent extremely high flows and would not supplement other structures in maintaining a navigable channel during normal flow periods.

(9) Low-level spur groins in crossings to provide additional contraction to 600 feet at low flows increased the depths over the crossings, but as some of them were transverse to the direction of flow they produced considerable backwater and undesirable shoaling upstream.

(10) Low-level longitudinal groins in crossings to provide additional contraction to 600 feet at low flows were effective in fixing the location and alinement of the crossings and produced some lowering of the controlling elevations over the crossings. As they were parallel to the flow, they offered practically no resistance.

Complete details of this model study are given in U.S. Army Engineer Waterways Experiment Station Technical Report 2–608, August 1962, entitled "Development and Maintenance of Navigation Channel, Arkansas River, Arkansas and Oklahoma."

A similar model study of the reach of river in the vicinity of a specific damsite on the lower Arkansas River is currently underway at the Waterways Experiment Station in Vicksburg, Miss., using a movable bed of granulated coal. Results of this study to date confirm the feasibility of developing and maintaining a deepend channel by means of a reasonable amount of contraction by dikes and revetments.

Flume Tests on the Armouring Effect of Gravel at St. Anthony Falls Hydraulic Laboratory

As mentioned previously, when the bed material load now carried by the river is reduced by the storage projects, the degradation process will be retarded by formation of an armouring layer of the coarser sized material forming the bed. This not only affects the riverbed elevation resulting from natural scour, but also is a

primary factor in the modified bed material load that is used to compute the equilibrium riverbed level as previously described. While an effort was made to account for the armouring effect of gravel in determining the loads, there is no generally accepted engineering analysis for the problem. Therefore, early in 1959, controlled flume tests were undertaken at the St. Anthony Falls Hydraulic Laboratory.

A sand-gravel mixture comparable to that on the lower Arkansas River was used in the flume studies. All tests were made by introducing clear water into the flume, or for zero sediment inflow. The studies were of two types: Small-scale exploratory tests and full-scale tests. In the small-scale tests observations were made of the degradation process on a bed initially composed of a homogeneous sand-gravel mixture. For each run the initial tractive force was greater than critical for all material in the bed, and as degradation proceeded, the slope flattened and the tractive force decreased to that of the critical value for the top layer of material that had accumulated on the bed when stability was reached. The initial tractive force was increased for each succeeding run, and the armour layer, approximately 1 grain-diameter thick, progressively coarsened.

In the full-scale tests an 18-inch sand bed was covered with an armour layer of 20-mm. gravel 3-grain-diameter thick, and the tests were largely devoted to observations of the natural development and healing of a scour hole in the bed, of phenomena at flow having a tractive force about critical for the armour material, and of the effect of an obstruction at the boundary on the gravel layer. The tests indicated that if excessive tractive force is localized and the mean value is less than critical for the armour, a scour hole will stabilize and armour itself; that if the mean tractive force is about critical for the armour material some of the fines will be leached out as the gravel is transported, the bed will be lowered, but the gravel blanket will remain essentially intact; and that if the tractive force is appreciably above critical for the armour, the gravel blanket will be disrupted and destroyed.

Results of Channel Studies

The estimated bed profiles for the 250-foot minimum width of navigation channel between Arkansas Post and Dardanella Dam under present conditions and for the computed equilibrium bed conditions with all works in place for the adopted plan of development are shown on figure 12. It will be noted that the equilibrium bed provides a minimum depth of 12 feet below the normal pools. The 8 low-lift locks and dams shown in the reach of the river between Dardanelle Dam and the vicinity of Arkansas Post compare with an earlier plan considered which included 13 low-lift locks and dams in the same reach with locations and pool elevations matched to the natural profile of the river. Thus, taking advantage of the degrading tendencies of the river after construction of the large dams made it possible to eliminate five locks and dams in this reach of river. Similarly, two locks and dams are being eliminated in the reach between Short Mountain Dam and the head of Ozark Reservoir. By a relocation of Ozark Dam site in conjunction with dredging and contraction, it now appears possible to eliminate a low-lift lock and dam previously considered between Ozark Dam and the head of Dardanelle Reservoir. Thus, a total of 8 low-lift navigation locks and dams out of a previous total of 18 are being eliminated by taking advantage of degradation with dredging, contraction works, and adjustment of damsites. As the cost of the dredging and contraction work is far less than the cost of locks and dams, a considerable overall reduction in project costs will be realized with added benefits through an increase in head for hydroelectric power generation at Dardanelle, Ozark, and Short Mountain Dams.

It is to be expected that a considerable amount of maintenance dredging will be required in the early years of the project, because of local shifting of material into the dredged channel from bars and from the undredged part of the main river channel. However, the maintenance dredging can be kept to a minimum and dependable navigation can be assured by dredging the initial channel to sufficient depth and width to provide space for deposition of the shifting bed material and by use of advance maintenance dredging. The greater is the extent of initial dredging, the less is the amount of material available for shoaling. As indicated before, navigable depths will improve and maintenance dredging requirements will diminish as time goes on as the effects of reduction in load progress downstream, as degradation continues, as a stabilizing armouring of the bed with coarser material develops, and as the supply of material in the streambed diminishes.

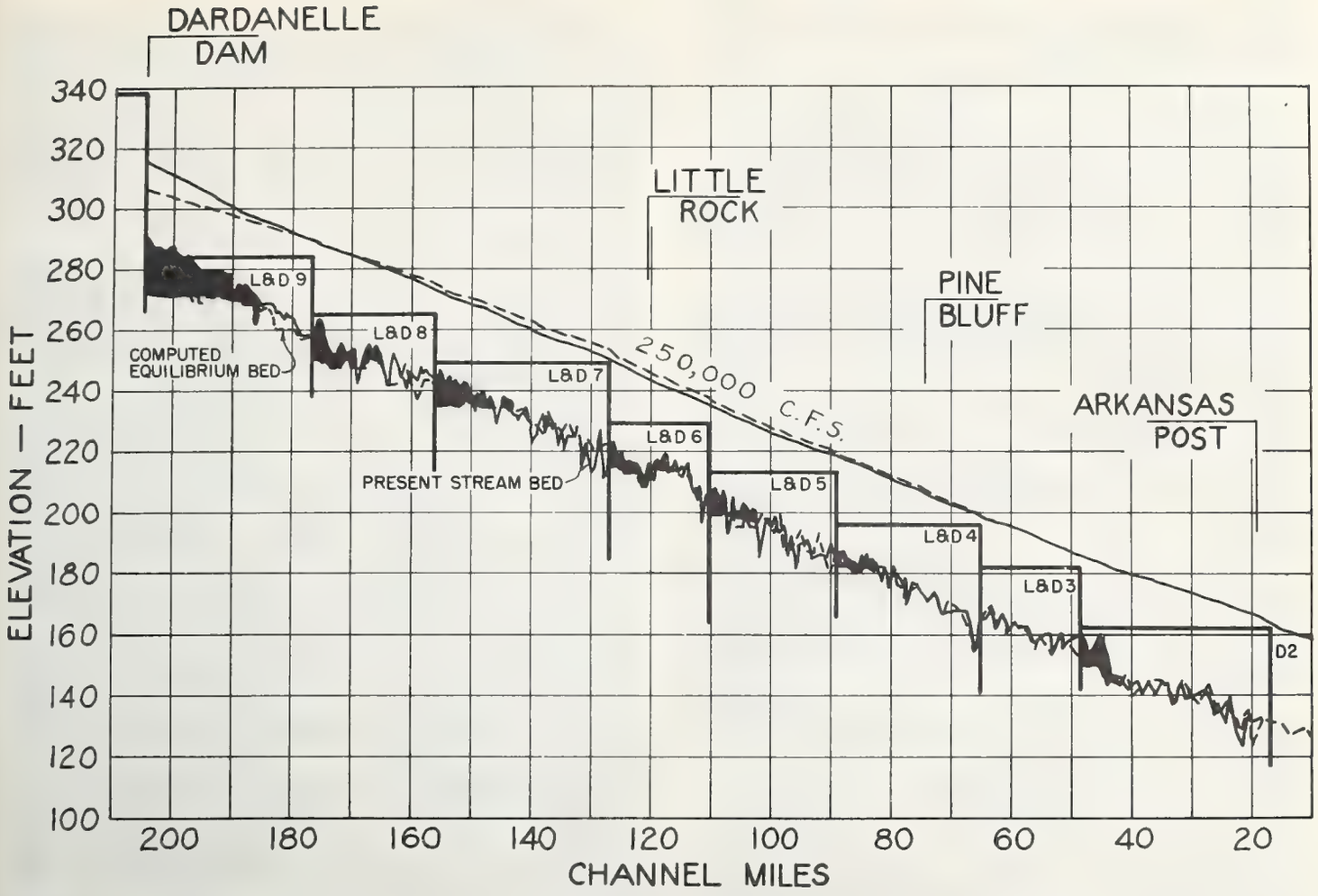


FIGURE 12. — Profiles for existing conditions and computed modified regime conditions between Arkansas Post and Dardanelle Dam.

CHANNEL RECTIFICATION STRUCTURES

[Paper No. 40]

By D. C. BONDURANT, *engineer, U.S. Army Engineer Division, Missouri River, Omaha*

Abstract

Channel rectification structures are utilized to prevent erosion of the banks, guide the flow along a desired alinement, restrict the flow to an effective waterway, or various combinations thereof. In most instances, these structures involve solid or permeable revetments or guide structures of varied types of materials and construction. There are few, if any, rational methods for design of such projects, and it is usually necessary to plan the layout wholly on the basis of experience and judgment. There is no attempt herein to present design criteria; rather, the areas of influence of various components are noted, the deficiencies in knowledge are pointed out, and the desirability of a comprehensive investigation to develop rational design is suggested.

Rectification Structures

Structures for channel rectification have been constructed of numerous types of materials and in a number of different ways. In general, each of these types of materials, varying from broken concrete and other trash — old auto bodies, rock, woven mattresses, piling or varied structural units — to solid paving or solid guide walls, should be adequate if properly applied at the right places. However, if improperly applied, they may be of no benefit or may even increase the damage they are meant to prevent. On the other hand, if applied properly but in excessive quantity or in locations where not actually needed, they result in an excessive waste of construction funds.

Rectification structures may be normally classified as either revetment or guide structures, although there may be a considerable overlap in their functions. Structures utilized

for stabilization of the bed of a channel will not be considered herein.

Revetments

Revetments may be placed either as an armoured blanket on a channel bank or as a guide structure to provide a smooth alinement and induce sedimentation to fill the bank line. The latter case will be considered with guide structures.

The purpose of a blanket revetment is to protect a channel bank against erosion. For this purpose, it should ideally possess the following characteristics:

1. It should be placed on a bank that has been properly alined and graded to a stable slope.
2. It should have adequate strength to resist movement by the flow or breakage by ice or debris.
3. It should be sufficiently tight to prevent leaching of the underlying soil materials.
4. It should be sufficiently flexible during construction to conform to any irregularities in the bank.
5. It should be mobile, in that it should settle into any areas of breakage or subsidence in a manner to be self-healing.
6. It should be permanent.
7. It should be susceptible to placement by reasonable methods.
8. Under certain circumstances, a hydraulically rough surface is preferable to one that is smooth.

The criterion for grading and alinement is most essential for larger streams, particularly those with easily erodible banks, but is beneficial in any case. The grading primarily provides a slope that can be more readily stabilized. A proper alinement eliminates eddies and deflections of the current. On smaller streams grading and alinement may not be economically feasible, and, if the banks are reasonably resistant, may not be necessary. The planner who will probably have noted instances of ungraded and unalined banks being adequately stabilized by trash dumps will have no guide lines except his own opinion of the resistant character of the banks.

Adequate strength is required in all cases. In the case of rock or similar materials, transport formulas, adjusted for bank slope factor, can be utilized to determine the size of the material required. A further adjustment might be used to reflect the keying action inherent in a well graded mixture of material sizes; however, there are no criteria available, and the uncertainties involved in transport computations make such adjustment inadvisable. Adequate strength of other materials, in place as revetment, is almost wholly a matter of judgment.

Leaching of the soils through the interstices of a revetment is a very common cause of failure, particularly if these soils are primarily sandy. Once a pocket has progressed to the extent that a local failure of the blanket occurs, the failure is apt to spread rapidly. Leaching can be inhibited or prevented by a tight blanket, a filter layer, or by a well graded mixture of rocks such that all voids are filled.

Most materials utilized for a blanket revetment permit sufficient flexibility in placement to conform to bank irregularities; however, any failure to do so will almost certainly result in ultimate failure of the blanket.

The question of mobility to the extent that a blanket will be self-healing will, if adhered to rigidly, eliminate many materials that might actually be adequate under the proper circumstances. As a general rule, however, it is an important criterion. Continuous pavement of concrete or asphaltic materals, for example, might be the most satisfactory armor under conditions where damage by ice or debris is not a factor, but once a break does occur at a location exposed to high velocity flow the armor is more apt than not to peel off in large strips. Hand-placed riprap, in addition to requiring expensive hand labor, tends to bridge over a failure until it becomes too extensive to be readily repaired. The most satisfactory material in this respect is dumped rock or even miscellaneous trash.

Permanence is not a problem with rock, broken concrete, and similar materials. Lumber and brush are relatively permanent where continuously submerged, but deteriorate rapidly where subjected alternately to wetting and drying. In locations where vegetation can be established once the bank is stabilized, the vegetation may adequately replace the deteriorated part of the revetment.

A reasonable method of placement would be defined largely in terms of the magnitude of the project.

The benefits of a hydraulically rough surface are not universal, and, in any event, are of value only within certain limits. They derive from the difference in velocity distribution as between rough and smooth boundaries; i.e., velocities immediately adjacent to the rough boundary will be less than those adjacent to the smooth boundary. In the Missouri River, for example, the high velocity flow normally occurring along the concave bank of a bend moves about 50 feet riverward after rock revetment is placed. As a concurrent feature, the deepest scour in the section occurs riverward of the toe of the revetment rather than at the toe. There are numerous examples of depths in bends of 15 to 20 feet below the toe elevation

of the revetment and several examples where this excess is greater than 30 feet. In none of these locations, however, has there been any undercutting or failure of the revetment.

An approximation of the range in which this effect would be of value can be made by computations of the two-dimensional velocity profile, but, since the rough boundary is generally sloping, the three-dimensional effect is more complicated. As the horizontal effect on the Missouri River is about 50 feet (with the revetment toe normally about 9 feet below the low water profile), it is probable that the benefits of the rough blanket surface would be limited to cases where the toe depth is not more than 15 feet below the low water profile. The greater turbulence generated at the rough surface increases the tendency for leaching; thus, a tight blanket becomes more important.

There are no universal design rules for blanket revetment. Normally the revetment should extend to the top of a low bank or to a level 8 or 10 feet above the low water profile; however, this latter value would vary with the size of the stream. Woven or other mattress is usually extended to the thalweg of the stream, but other types of revetment are usually stopped at the intersection of the bank with the streambed. If the project is of sufficient magnitude, it is advisable to either excavate a trench to extend the mattress several feet below the normal bed or to add excess toe material that can settle to heal undercutting. In locations where the flow hits the bank at an angle greater than about 30°, a mattress extending to the thalweg is advisable, regardless of the type of upper bank blanket.

Permeable Training Structures

Permeable revetments and permeable training structures are similar in purpose and construction, their primary function being to guide the flow to a desired alinement and, concurrently, to cause sedimentation in a desired location landward of the structure. They have been constructed of piling, fencing, steel or timber cross frames (jacks), brush, and other available material. There are two schools of thought insofar as the design principles of such structures are concerned: (1) that the units are considered as roughness elements that cause a reduction of the velocity of the flow through the structure and (2) that the structure should primarily guide the flow while concurrently permitting a part of the flow to pass through into a relatively quiescent area to deposit its sediment load. Both are correct, for the two functions are interrelated, and the one that predominates is a matter of the overall layout.

The most rational analysis of the action of a permeable structure is, perhaps, that presented by Einstein [1] — "When the division takes place between the water masses which move along the jacks and those which pass through the line of jacks, all the water has entered a local zone near the structures of slightly higher pressure. In entering this zone, all water particles lose, according to momentum theory, a given constant part of their velocity." Einstein computed that if the jacks were fully effective it would require 140 jacks (composed of 18 foot timbers) to turn a flow of 50,000 c.f.s., flowing at a velocity of 5.7 feet per second, around a 90° bend. He estimated, however, that since the jacks are only partially effective due to their splitting action, the actual requirement would be in excess of 500 units. He concluded that secondary effects, particularly those of the sand deposits behind the jacks, were a major part of their function.

For a different attack, Carlson and Enger [2] measured the head loss in a model. Actually, it can be observed in the field that a permeable structure which is essentially parallel to the flow acts as a permeable rough boundary and that as the angle to the flow increases, this effect decreases and the structure acts more as a roughness element in the line of the flow. In either event, there appears to be a pressure gradient as discussed by Einstein.

The preceding discussion cannot be said to solve anything in particular except to point out the fact that the designer knows very little about the tool with which he is working. The lack of knowledge may well result in either an unsuccessful project or one in which a necessarily conservative approach generates excessive costs. The average project utilizing permeable structures will include guide lines to assist in turning the flow and to cause a reduction in the velocity of the flow passing through it. Between this frontal line and the existing bank there will be tielines, either solid or permeable, which will be expected to reduce further the velocity of the flow in the sedimentation area, to prevent the buildup of a secondary flow system, and to return flow from the sedimentation area to the channel. In projects that have continued over a period, experience will have developed criteria for the optimum angles of the guide lines, the size and spacing of the units therein, and the number and locations of the tielines required, but this experience will not be too helpful if transferred to a system on an entirely different scale.

[1] EINSTEIN, H. A. REPORT ON THE INVESTIGATION OF THE FUNDAMENTALS OF THE ACTION OF RIVER TRAINING STRUCTURES. Calif. Univ. [Rpt. not published.] 1950.

[2] CARLSON, E. J., and ENGER, P. F. USE OF STEEL JETTIES FOR BANK PROTECTION AND CHANNELIZATION IN RIVERS. Presented at Amer. Soc. Civil Engin., Hydraul. Div. Mtg., August 1956, Madison, Wis.

Groins

Groins are generally structures placed normal to the current or at moderate upstream or downstream angles thereto, and may be likened to some extent to the tiebacks in a permeable system without the frontal lines. As such, they are not considered to be desirable if constructed as short segments along a straight or concave bank, but there have been some recent indications that a series of groins, or tieback, structures with a downstream extension (L-head) may permit deletion of a part of the frontal line. In other words, instead of being continuous the frontal line might extend only one-half or two-thirds of the distance from one tieline to the other.

Any structure extending into and normal to the flow will have several generally undesirable effects. The primary such effect is the formation of a relatively deep scour hole immediately at and downstream from the outer end of the structure. A second such effect is the ever-present danger of destruction of the root end of the structure, and a third is the fact that a system of groins along a straight or concave bank can increase the overall roughness coefficient of the reach by as much as 25 percent. When a downstream extension is placed at the outer end of the structure, however, the structure begins to approach the characteristics of a frontal system with tiebacks. Some field trials under way indicate that, if the downstream leg extends to one-half to two-thirds the distance to the next structure and if the structures are properly placed in a straight reach or flat bend, both the deep scour and the excessive roughness will be partially or perhaps wholly eliminated. This means only that adequate guidance might be provided by a discontinuous rather than a continuous frontal line; however, the evidence to date is inadequate to provide any rational proportions or justify use except on an experimental basis. There is little doubt that even a short L-head improves the action of a groin, as such.

Research Needs

An observer noting numerous examples of successful revetment and training structures might have reason to consider the foregoing discussion unnecessarily pessimistic. He might also recognize that the ultimate success of a permeable system is dependent upon auxiliary factors such as the effect of the sediment deposit itself and the vegetation that becomes established thereon, and, so recognizing, discount any need for refinement in the initial structure. He might even be right on both counts.

The planner who must prepare the project, however, finds that there is a general similarity in the layout of such projects, but beyond that point he is largely dependent upon intuition. This is hardly a proper concept of engineering; therefore, several areas in which some investigation would be beneficial are suggested.

In terms of blanket revetment, the most fruitful research would appear to be in the area of the required strength; i.e., for example, the size of rock required, with and without grading, on various bank slopes, to resist a given flow velocity. The required thickness of blanket also needs analysis.

There are uncertainties as to the proper depth to which a revetment should extend. The "toe trench" rock revetment utilized on the Missouri River for 20 years without a failure of consequence is also used successfully on the Arkansas River, yet the writer has been informed that on the latter stream there will frequently be a slip in the original toe rock and that a second step to add additional rock is utilized to obtain a permanent structure. To date, the reason for this difference has not been established. The general dimensions are similar in both cases.

It has also been noted in a small model that the existence of large dune formations leads to excessive scour, owing to deep eddy action at the toe of the dune. This question is probably of academic interest only, since dunes of the relative size noted are not apt to occur in natural streams, but it should be investigated further.

Investigation of the basic action of permeable guide structures without regard for secondary influences are needed, for these secondary influences cannot be effective until engendered by the basic structure. Such an investigation would be required to measure the varied effects of the structure in all positions from normal to the flow to parallel to the flow, and would need to consider the size and spacing of units in relation to flow area as well as frictional effects. A complete velocity pattern should be developed for analysis by momentum or other hydraulic theory; the ultimate purpose is to permit a theoretical analysis to determine the most economical and effective layout and spacing of the frontal line and the various tieback lines required in the deposition area.

The transport function should be investigated further, to determine the proportions of sediment diverted under varied arrangements of the frontal line and varied conditions of transport or of sediment load in the stream.

There should be a simultaneous empirical analysis, since this approach would probably yield the most immediately useful results. The relative effect of the secondary influences would be a necessary study here, as the designer

would, in most cases, desire to reduce the theoretical structure somewhat in accordance with anticipated secondary actions.

In one recent instance,[3] a model study was conducted for an individual project design. Considering the fact that the initial cost of structure in the field can vary from a minimum of perhaps $5 per lineal foot of structure to well over $30 per lineal foot, it seems obvious that the elimination of a single tieline or of a hundred feet or so of a frontal line would more than pay the cost of the model. It is surprising that more such analyses have not been made.

Mention has been made of field investigations of an L-head structure. Actually, the L-heads were added to a series of perpendicular dikes in an effort to improve their action and eliminate some of their deficiencies. In the locations tested; i.e., the concave bank of a flat bend, the improvement has been impressive. The cost of prototype testing, however, will eliminate extension beyond the point of satisfactory action in a particular location. The questions of optimum height and relative length for general application will remain unanswered insofar as the field installation is concerned, but they could be resolved in a model at moderate cost.

Unquestionably many other facets of the problem could be discussed; however, if the preceding material develops some thought and engenders some realistic analyses, the writer's purpose will have been served.

THE RIO GRANDE COMPREHENSIVE PLAN IN NEW MEXICO AND ITS EFFECTS ON THE RIVER REGIME THROUGH THE MIDDLE VALLEY

[Paper No. 41]

By Robert C. Woodson and John T. Martin, *engineers, U.S. Army Engineer District, Albuquerque, N. Mex.*

Synopsis

The Rio Grande comprehensive plan in New Mexico consists of a system of reservoirs on the main stream and tributaries. The reservoirs are located upstream from the head of a reach of about 180 miles of rehabilitated floodway through the Middle Valley.

Sediment detention and regulation of floodflows by the reservoirs are expected to prevent further aggradation and inhibit excessive degradation through the rehabilitated floodway.

The comprehensive plan in New Mexico consists of a system of reservoirs (Abiquiu, Jemez, Cochiti, Galisteo) on the Rio Grande and tributaries at and near the head of the Middle Valley and the rehabilitation of the Rio Grande Floodway constructed by the Middle Rio Grande Conservancy District in 1935. The Rio Grande Floodway confines the river throughout most of the Middle Valley, which comprises a reach approximately 180 miles long. Locations of the elements of the plan are shown on figure 1.

The reservoirs are for flood and sediment control. The levee improvements and channel stabilization are necessary in the interest of flood control, major drainage, and water salvage.

In the Cochiti-Rio Puerco reach of the floodway, the Rio Grande occupies a wide shallow channel between the levees of the floodway. The channel has no banks, and the average level is generally at or above the level of the areas behind the levees.

The levees constructed as a part of the floodway by the conservancy district were subject to attacks by the river. Early attempts to protect them by means of pile jetties demonstrated the need for a more effective system of levee protection. Deterioration of the floodway due to aggradation and the high losses of water which were occurring indicated the need for regulation of floodflows, sediment retention, and channel stabilization.

In 1948 the Corps of Engineers and the Bureau of Reclamation jointly recommended the comprehensive plan of improvement for the Rio Grande in New Mexico. The dams would be constructed by the Corps of Engineers and the floodway rehabilitation would be constructed jointly by the Corps of Engineers and the Bureau of Reclamation. The plan was authorized for construction in 1948, except for a main stream reservoir that was to provide control of floods originating principally in Colorado.

A restudy of the problem of main stream control was made by the Corps of Engineers in 1958 and two additional reservoirs were authorized by the Flood Control Act of 1960, one on the main stem at Cochiti at the head of the Middle Valley and the other on Rio Galisteo.

The Jemez and Abiquiu Reservoirs and the floodway improvements in the Cochiti to Rio Puerco reach are constructed, and the Cochiti and Galisteo Reservoirs are in the preconstruction planning stage. The Cochiti to Rio Puerco

[3] Carlson, E. J., and Dodge, E. A. control of alluvial rivers by steel jetties. Amer. Soc. Civil Engin. Proc., Waterways and Harbors Div. Jour. 88 (WW4): 53-81. Nov. 1962.

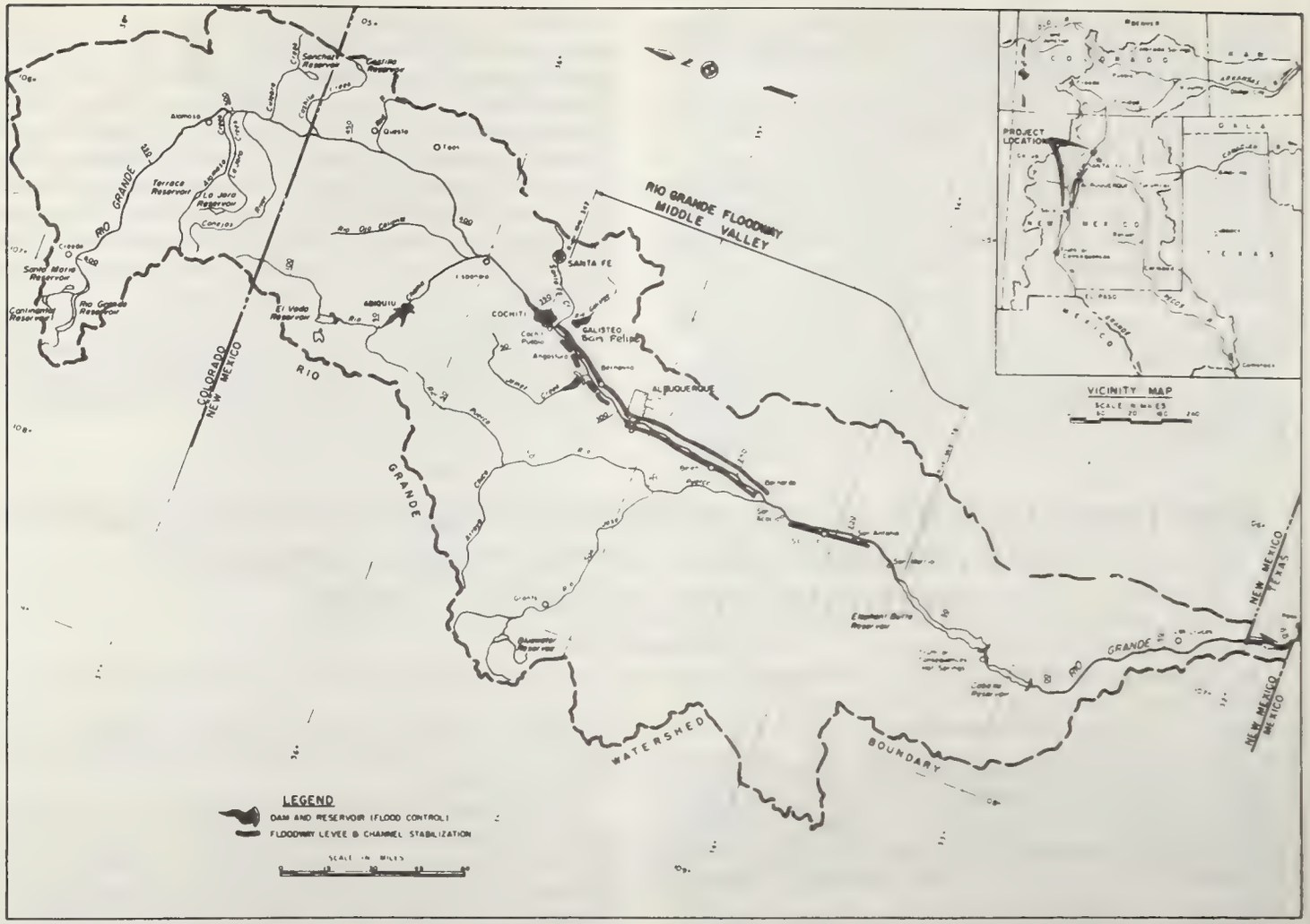


FIGURE 1. — Rio Grande floodway, Middle Valley, N. Mex.

reach comprises about 105 miles of the total floodway length. This discussion primarily concerns the plan of development and its effects in this reach.

The Middle Rio Grande Conservancy District was organized in 1925 under the laws of the State of New Mexico for the primary purposes to improve drainage, irrigation, and flood control for about 128,000 acres of land, including urban population centers within the Middle Valley. The district was subdivided into four divisions: Cochiti, Albuquerque, Belen, and Socorro. The principal works were begun in 1930 and completed in 1935. Because of the inability of the district to finance a flood control system involving a number of reservoirs, the basic flood control element adopted for the Middle Valley was a floodway. Figure 1 indicated the extent of the floodway through the Middle Valley.

In general, the levees were located parallel to the then existing meander pattern of the river. Because the bends in the river are moderate, no significant attempts were made to straighten the river by cutoffs. The width of the floodway between levees is about 1,500 feet and the average height of the levees is about 8 feet. The adopted design flow for this reach was 40,000 cubic feet per second except at the city of Albuquerque where extra height of levee was provided for a design flow of 75,000 cubic feet per second.

Below the Rio Puerco the levees were constructed only on the west side of the river to the village of San Antonio, a distance of about 50 river miles. The design capacity of the floodway in this reach was 50,000 cubic feet per second.

The method of computing aggradation or degradation consisted of determining the volumetric changes in the valley between surveys of the range lines from 1936 to 1953, inclusive. The computations were made for the channel area, the flood plain area, and the combined channel and flood plain area. The channel area consists of that part of the floodway that does not contain vegetation but is characterized by

sandbars and low flow channels. The flood plain area is designated as the remaining areas between levees or bluffs subject to overflow only during periods of high flows. This area usually contains growths of trees, shrubs, grass, and weeds of varying density.

Volumes were computed by multiplying the average depth of ground surface below an established base plane at each range by the area between ranges. The difference between volumes of any two surveys represents the change in volume by aggradation or degradation that occurred in the time interval between the surveys. Some adjustments were made to the surveys prior to the 1952–53 surveys because of differences in methods used and in range numbering. A summary of the results of the aggradation and degradation study for the greatest period (1936–53) between surveys is shown in table 1.

TABLE 1.—*Aggradation and degradation, Cochiti to San Antonio, N. Mex., 1936–53*

River and division		Plane area	Net change[1]	Rate of change[1]
		Acres	*Acre-ft.*	*Ft./yr.*
Cochiti (22.5 river miles)	Channel	2,057	−1,515	−0.043
	Flood plain	2,049	+1,810	+ .052
	Channel and flood plain	4,106	+ 295	+ .004
Albuquerque (39.9 river miles)	Channel	4,286	+1,153	+ .017
	Flood plain	4,183	+4,582	+ .068
	Channel and flood plain	8,470	+5,735	+ .042
Belen (52.6 river miles)	Channel	7,704	−2,575	− .020
	Flood plain	5,575	+3,284	+ .035
	Channel and flood plain	3,279	+ 709	+ .031
Socorro[2] (29.0 river miles	Channel	4,203	+2,614	+ .037
	Flood plain	4,264	+1,520	+ .021
	Channel and flood plain	8,467	+4,134	+ .029

[1] + = aggradation; − = degradation.
[2] Terminates at San Antonio.

The period used in table 1 begins shortly after the completion of the floodway and the diversion dams constructed by the Middle Rio Grande Conservancy District and reflects the full effects of these structures. It also includes the high flow years of 1941 and 1942.

From table 1 it can be seen that the Rio Grande within the Middle Valley is still an aggrading stream, and construction of the floodway has not halted aggradation. Aggradation has occurred in amounts varying from practically zero in the Cochiti Division at the upstream end of the valley to a maximum of about 0.04 of a foot per year in the Albuquerque Division. The indicated channel degradation that is occurring in the Belen Division is not believed indicative of a long-term trend. The Rio Puerco enters the Middle Valley at the lower end of this division. The Rio Puerco is the largest sediment-contributing tributary of the Rio Grande in New Mexico and contributes well over 50 percent of sediment load entering the valley. The indicated channel degradation is believed to be the result of the river cutting through a large sediment plain deposited in the Rio Grande by the Rio Puerco below the mouth during the major flood of 1929. It is known that during this flood a large part of the flow came from the Rio Puerco, together with an enormous flow of sediment. The 1929 flood occurred before the beginning of the period of measurements reflected in table 1. In the event of another flood similar to the flood of 1929, which can be reasonably expected, it is believed this trend toward degradation would be sharply reversed. The river channel at the present time is aggrading above the junction of the Rio Puerco about the same rate as it is degrading below.

In all, about 190 miles of levees were constructed, together with a small number of levee protection works. The levees, together with the natural bluffs along the river, confined the river within the floodway throughout the Middle Valley. In addition to the flood control works, four major diversion dams and several hundred miles of drains, canals, and related works also were constructed.

The work of the conservancy district did much to improve distribution of irrigation water and reduce waterlogging of lands within the district. However, after the major flood year of 1941, it was clearly evident that the floodway would not afford the protection that it was designed to provide. The mean daily peak of this flood was only 22,500 cubic feet per second; however, this long duration flood covered a period of about 2 months with flows about 20,000 cubic feet per second for 5 days and above 10,000 cubic feet per second for 43 days. The levees were breached in 25 places and active flood fighting was necessary at 80 different locations along the river. Extensive flood damage was experienced.

Subsequent to this flood, the Corps of Engineers made a comprehensive investigation for flood control along the Rio Grande in New Mexico with a view to developing a more adequate plan for flood and sediment control. Because of the effects of the river regime on the agricultural economy and the need for further improvement of the irrigation works of the conservancy district, the U.S. Bureau of Reclamation began cooperative studies with the Corps of Engineers. Emphasis was placed on determination of aggradation or degradation that had occurred in the Middle Valley after the construction of the works of the conservancy district. Considerable speculation had existed as to whether or not the trend toward aggradation was continuing and to what extent it had taken place.

Fortunately in 1936, shortly after the Rio Grande floodway was placed in operation by the Middle Rio Grande Conservancy District, the Soil Conservation Service of the U.S. Department of Agriculture established and surveyed river cross sections or ranges across the floodway within the entire Middle Valley. The ranges were to be resurveyed at later dates so that changes in riverbed elevations could be determined. These ranges were located approximately at 2-mile intervals. Additional ranges were established by the Corps of Engineers in 1944, which reduced the interval between ranges to one-half mile. The ranges were surveyed in 1936, 1940, 1941, 1942, 1944, and 1952-53. A cooperative study of aggradation and degradation based on these surveys was made by Bureau of Reclamation and the Corps of Engineers.

The rate of change for the period covered by the surveys within the Albuquerque division is illustrated in figure 2. The high runoff years influenced the rate of change in both the channel and the flood plain. The occurrence of a high runoff year like 1941 was instrumental in keeping the 1936–53 aggradation rate in the channel to a low amount while increasing aggradation rate in the flood plain. Another factor that is partially shown on figure 2 is the influence of the relative order or cycle of flows upon the rate of change. An example is the change that resulted from the 1942 runoff, which was a high flow year after the still higher flow in 1941. The aggradation rate in the flood plain was about the same for 1941 and 1942, whereas the channel change was practically zero. It is doubtful that the change in 1942 would have been the same if it had followed a series of low runoff years. The conclusion reached, from the plotting of figure 2, is that within the limits of the Middle Valley there is a definite relation between the annual runoff and the aggradation or

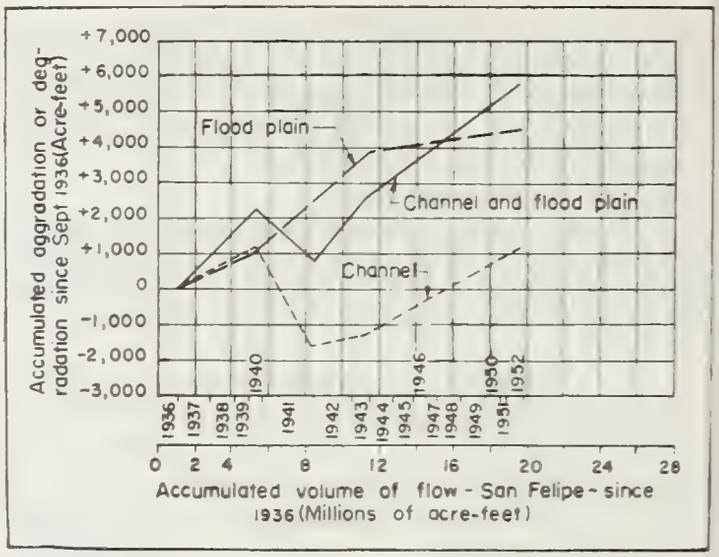


FIGURE 2. — Rate of change — Albuquerque Division (1936-53).

degradation in the channel and flood plain. A further fact revealed by the surveys, but not shown in table 1, is that there has been no material net change in the relation of areas of channel and flood plain since 1936.

Simultaneously with the survey of the river ranges in 1936, the Soil Conservation Service began an extensive program of sediment sampling that has been carried on through the cooperation of other Federal agencies, including the Corps of Engineers and the Bureau of Reclamation. The purpose of the sediment sampling program was to determine the origin, quantities, and characteristics of sediments transported by the Rio Grande. When this program was begun, except for the station at San Marcial, less than 20 samples of water had been analyzed for sediment content. To date about 70 thousand integrated samples from the main stem of the river and tributaries have been collected and analyzed. The majority have been collected within the Middle Valley.

Flows are very turbulent which makes suspended sediment samples from the Rio Grande more nearly indicative of the total load than similar samples from other rivers. The suspended sediment samples contain a considerable amount of sand, which increases with increases in rate of flow. For example, the sand load in the samples at Cochiti averages about 3 percent of the total load from flows less than 500 cubic feet per second, and 34 percent in 32 samples from flows between 2,000 and 3,000 cubic feet per second. The amounts at Bernalillo are 17 percent in 29 samples from flows less than 500 cubic feet per second, and 29 percent in 16 sam-

ples from flows between 2,000 and 3,000 cubic feet per second. Table 2 shows sediment flows through 1954 at the principal stations within the Middle Valley.

TABLE 2.—*Discharge and sediment summary, 1936–54*

Division and station		Drainage area	Average annual discharge	Annual sediment discharge
		Sq. mi.	*Acre-ft.*	*Acre-ft.*
Cochiti division	Cochiti	11,440	1,212,400	2,930
Albuquerque division	Angostura	12,941	1,165,900	3,342
	Bernalillo	14,059	1,089,000	3,921
	Albuquerque	14,502	1,054,000	3,764
Belen division	Isleta	14,737	1,127,000	3,925
	Belen	15,291	1,039,800	3,616
	Bernardo	15,929	1,017,800	3,446
Socorro division	San Acacia	23,623	1,092,150	10,182
	San Antonio	24,460	1,058,900	9,645
	San Marcial	24,558	1,061,500	7,626
	Head Elephant Butte Reservoir	24,600	1,063,900	5,838

The significant increases in sediment flow from Angostura to Bernalillo are due to the inflow from Jemez Creek, which enters between these two stations. The large increase between Bernardo and San Acacia is due to the Rio Puerco, which enters between these stations. The sediment inflow at Cochiti is derived principally from the Rio Chama.

The general trend toward aggradation indicated by the surveys of river ranges also is confirmed by the sediment discharge compilation in table 2. For example, in the reach between Bernalillo and Bernardo, which is between two of the major sediment contributors to the Middle Valley, Jemez Creek and Rio Puerco, the annual inflow of sediment is 3,923 acre-feet and the outflow is 3,446 acre-feet, despite an approximate increase in drainage area of 1,900 square miles.

The plan for rehabilitation of the floodway follows in general the original levee alinements. However, the design capacities for the floodway were materially reduced except in the reach below the Rio Puerco. This is due to the high degree of control provided by the reservoirs above the Rio Puerco. The design capacity above the Rio Puerco is 20,000 cubic feet per second except at Albuquerque, where the capacity is 42,000 cubic feet per second. Below the Rio Puerco, the design capacity is 60,000 cubic feet per second.

The most noteworthy improvement of the floodway is the provision of a rectified channel within the floodway from Cochiti to the Rio Puerco. The purpose of this channel is to provide protection to the levees and decrease the existing channel area in the interest of reducing water losses and improving water and sediment transport capacity. The original river channel, which averaged about 800 feet in width, was narrowed to a uniform width of about 550 feet. This was accomplished by an arrangement of jetties projecting from the levees on both sides of the river to the channel and lines of jetties paralleling the channel.

An interesting feature of the channel rectification work from a sedimentation standpoint is the use of the Kellner jetty system. In the past many types of levee protection and channel rectification works have been used on the Rio Grande. These have been generally of the more rigid types, utilizing timber piling in combination with woven wire or wire baskets of brush or rockfills. Jetties employing piling have proved generally unsatisfactory for conditions prevailing on the river. Channel scour often undermines the piles and destroys their retarding ability. The Kellner jetty system is particularly adapted to heavy silt-laden streams subject to considerable variation in channel scour during periods of high riverflow. These steel jetties are composed of permeable and extremely flexible multiple-units that readily conform to channel scour. They permit sediment-laden flows to penetrate the jetty where velocity and energy are reduced. Sediment is deposited behind the jetty, thereby building up an area or bank behind the jetty. Vegetation grows rapidly on the accumulated sediment, which further increases the retarding effects. The system is particularly adapted to the protection of levees, riverbanks, and bridge abutments.

The single or standard unit of the system is called a "Jack" and connected together they form a jetty. A series of lines of jetties form a jetty field. The layout of a jetty field involves two types of lines of jetties, the diversion lines and the tiebacks, or retard lines. The diversion lines, usually two in number, are placed roughly parallel to the levee or bank of the river. The retard lines extend from the diversion line back to the levee, or bank. They are usually placed at an angle of roughly 67.5° to the diversion line

at any desired spacing between lines. As installed on the Rio Grande, the spacing of retard lines at river bends is usually about 250 feet. Single diversion lines and wider spacing of retard lines are used in less critical areas.

The Kellner Jetties Corp. pioneered the development of the system. General principles usually followed in the layout of a jetty field and the design at curves are based on model studies of the retarding effects of the jetties and of experience. More detailed information is contained in the Corps of Engineers' "Report on Measures for Bank Protection, Use of Kellner Jetties on Alluvial Streams," June 1953, revised August 1962, and the Bureau of Reclamation's publication, "Use of Steel Jetties for Bank Protection and Channelization in Rivers," by Enos J. Carlson and Phillip F. Enger, Hydraulic Engineers. This type of jetty is comparable in cost to the more rigid types, but the effective life is much longer. There are installations with service records of 30 years with a reasonable life expectancy of 50 years. Figures 3 and 4 illustrate the standard unit and its use in a typical jetty field utilized by the Corps of Engineers and the Bureau of Reclamation on the Rio Grande floodway. Figure 5 shows the progressive effectiveness of a jetty field in causing deposition and growth of vegetation to change the regime of the river.

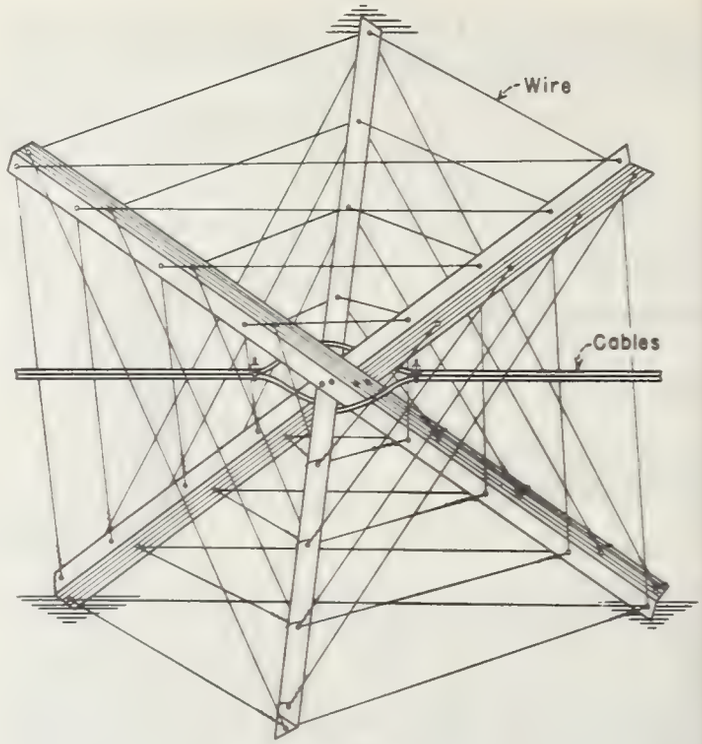

Wire
Cables
NOTES:
(1) Unit is usually 16'x 4"x 4"x ¼" angle iron laced with No. 6 wire.
(2) Cables are usually ¾" ϕ.
(3) Above unit is placed 12½' on ctrs.

FIGURE 3. — Standard unit Kellner jack.

Studies were made by the Bureau of Reclamation to estimate the combined effects of construction of the flood and sediment control reservoirs and the channel rectification work on the river regime within the Middle Valley. The effects on floodflows were readily determined by routing studies. The effects on channel aggradation and degradation were more complicated; however, the data collected from the sediment studies and survey of the river ranges were believed sufficient for a reasonable estimate of anticipated effects. The procedure followed in general is as follows. The river was divided into convenient reaches that were treated as units with similar characteristics within the reach, such as slope, width, and riverbed material. From the suspended sediment sampling and flow records, the suspended load was determined. Bedload, or the load not measured by sampling, and the total load were then determined by the use of the modified Einstein method developed by the U.S. Geological Survey and the Bureau of Reclamation. This was the basic condition without dams or rectified channel.

If the same reaches but with modified condition of rates of flow and sediment inflow due to the construction of the dams and the rectification work are used, the capacity of the reach to transport sediment was determined by application of the Lane-Kalinske formula for suspended sediment load and the average of the Kalinske and Schoklitsch formulas for the unmeasured load. The results were then adjusted by a factor to conform to the results obtained by the application of the modified Einstein method. This factor was determined by comparing the results obtained with the use of these formulas in the unmodified conditions with the results in the same reaches obtained by the modified Einstein method. Samples of streambed material were obtained at the river ranges and from these, composition of the riverbed material in each reach was estimated for several divisions of grain sizes, each of which is subject to transport at different rates of flow. The amounts of sediment entering the reach were subdivided in a similar fashion. Assumptions were made as to depths of scour of streambed material. The sizes smaller than sand were assumed to pass through the reach. With the ability of the stream to transport the various sizes and the total amount of the various sizes in the riverbed and entering the reach available for transport known, outflow from the reach was determined. This process was repeated for each successive year and the net differences in inflow

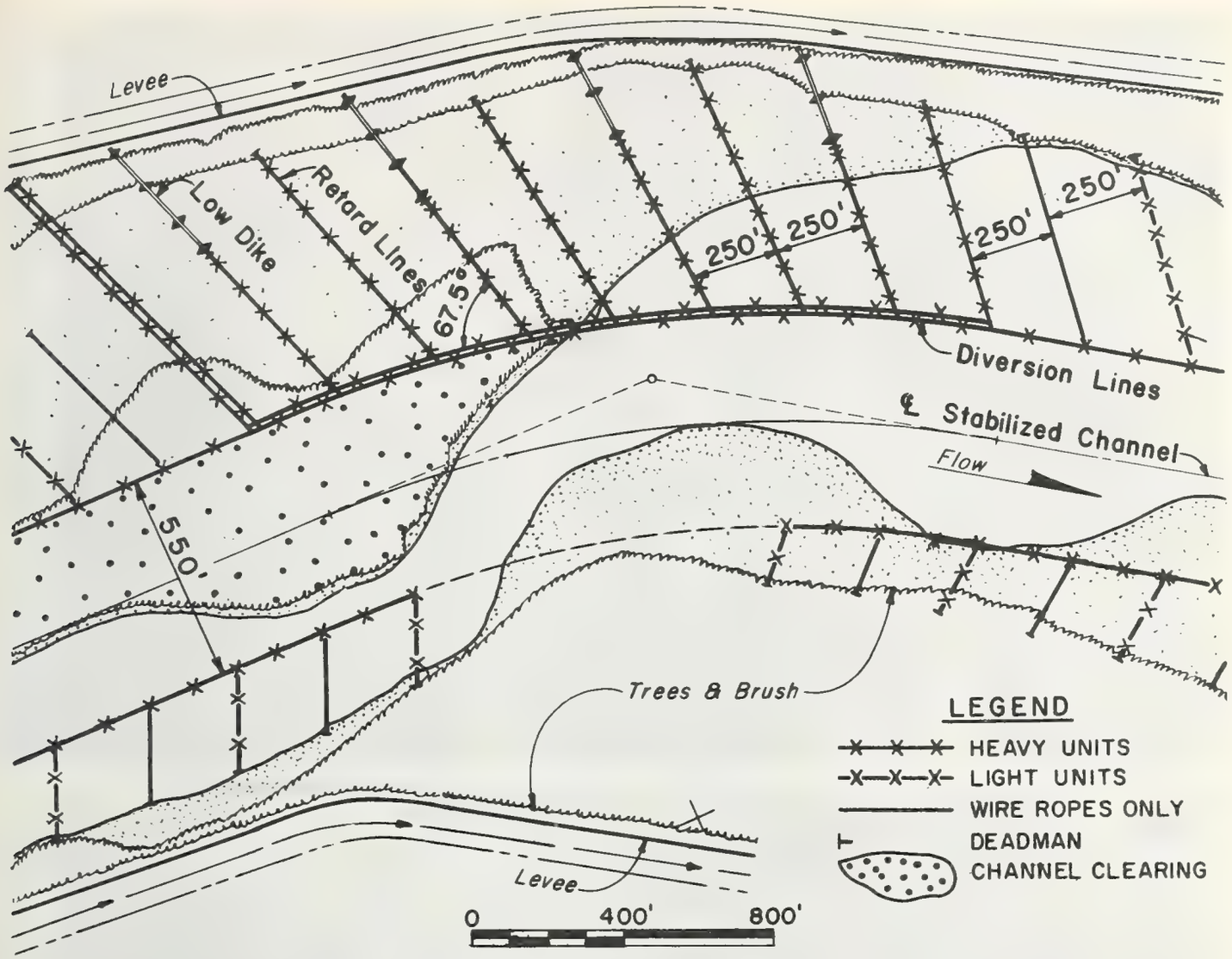


FIGURE 4. — Typical Kellner jetty field—Rio Grande, N. Mex.

and outflow of sediment indicated aggradation or degradation for the reach.

The results indicate that the sediment inflow at Bernalillo will be reduced by the reservoirs about 75 percent after about 20 years and that the present trend of aggradation will be reversed. Indications are that degradation will begin at Cochiti and will extend downstream at least to the Rio Puerco. Progressive coarsening of the streambed will inhibit degradation. It appears unlikely that the stream will degrade more than about 3 feet in the Bernalillo to Albuquerque reach. It is probable during the first 10 to 15 years after the reservoirs are all completed and in operation that considerably more degradation will be apparent in certain locations. This will be due to the confinement of the clear water releases by existing bars, etc., while the 550-foot channel is being flushed out. After this flushing has taken place, the energy of the clear releases will be more evenly distributed over the full width of the channel; flows will have less sediment transportability. As a result, it is anticipated by the authors that this will tend to inhibit excessive degradation at any one place.

The Corps of Engineers and the Bureau of Reclamation have a cooperative agreement to share the costs of periodic photogrammetric resurveys of the entire Middle Rio Grande Valley. The initial condition survey was made during the spring of 1962. Low level contact prints, controlled aerial mosaics, and cross sections to a scale of 1 inch equals 500 feet at approximately 500-foot intervals will be obtained. These data will be analyzed to check the theoretical estimates of sediment movement, including location of aggradation and degradation. Also, the photographs will show the progressive changes in vegetal growth.

FIGURE 5. — Emergency flood control work on Rio Grande near Bernalillo: *A*, December 1952, looking downstream prior to installation of bank protection works; *B*, August 1953, at the same location after installation of Kellner

jetty field in June 1953; *C*, September 1955, about 2 years after completion of emergency work; *D*, April 1958, about 5 years after completion of bank protection works.

METHODS AND CRITERIA FOR BANK PROTECTION ON THE LOWER COLORADO RIVER

[Paper No. 42]

By FRANK K. ILLK, *hydraulic engineer, Bureau of Reclamation*

Introduction

Bank and bed stabilization along the Lower Colorado River has a history extending back over 50 years. The Bureau of Reclamation first became actively engaged in the problem in connection with the revetment of the levee system protecting the Yuma project near Yuma, Ariz. This work was accomplished in the early years of this century.

Other individuals and agencies, notably the Palo Verde Irrigation District, were also engaged in bank stabilization activities at an early date.

The closure of Hoover Dam in 1935, with its high degree of control over the flow of the Lower Colorado River, instituted a new river regime in which channel stabilization has become increasingly important (fig. 1). The necessity for dealing with the problems inherent to the new regime led to the creation of the Colorado River Front Work and Levee System by Public Law 469 — 79th Congress, Second Session. The basic responsibilities of the Bureau are set forth in the following quotation from the Act:

(a) operating and maintaining the Colorado River Front Work and Levee System in Arizona, Nevada, and California; (b) contructing, improving, extending, operating, and maintaining protection and drainage works and systems along the Colorado; (c) controlling said river, and improving, modifying, straightening, and rectifying the channel thereof; and (d) conducting investigations and studies in connection therewith.

Under the authority of this Act the Bureau, through Region 3, has engaged in a variety of activities planned to produce eventually a stable channel from Hoover Dam to the Mexican Boundary.

River Characteristics

Historically, the Lower Colorado River carried enormous loads of sediment. Practically all of the sediment originated in the upper basin above the present site of Hoover Dam. The lower basin was apparently actively aggrading, with the sediments of the river gradually building up the bed and banks of the stream and raising the floor of the canyons and valleys.

This natural regime was abruptly terminated by the construction of Hoover Dam and the formation of Lake Mead. This lake has ample capacity to store the sediments originating in the upper basin and as a result, the release from Hoover Dam is almost completely sediment free. The introduction of this clear water into a channel which had been graded and molded by flows containing sediment concentrations of up to 50,000 parts per million caused a rapid readjustment in the channel characteristics.

A factor that causes the Lower Colorado River and its problems to be somewhat unique is its location within one of the most arid regions in the United States. In the entire 440 miles of river from Hoover Dam to the Gulf of California there are only two perennial streams, the Bill Williams and the Gila, and neither of these normally contribute an appreciable amount of either water or sediment inflow. As the result of no inflow and the diversion of water for irrigation at many points along the river, the flow diminishes downstream from Hoover Dam until, more often than not, it dries up completely at or just below the southerly International Boundary.

The sediment pattern is somewhat more complex. Since there is essentially no sediment inflow to the river, all of the sediment that moves through the Lower Basin is derived by scour from the alluvium of the bed and banks. Adding to the complexity is the effect of the various dams along the lower river. Below Hoover Dam, the river is divided into a series of reaches, or divisions, by Davis, Parker, Headgate Rock, Palo Verde, Imperial, Laguna, and Morelos Dams. These dams affect the sediment pattern more or less according to their functions. Storage dams such as Davis and Parker intercept any sediment picked up by the river and reintroduces clear water into the river downstream. The diversion dams mark the points of depletion of the flow and in some cases increase the sediment load downstream by diverting larger portions of water than of sediment or by returning sediment to the river. Each of the reaches between these dams differ somewhat in the character of the channel and in the control problems associated with these conditions.

The unique circumstance that practically all of the sediment load of the Lower Colorado River is derived from the bed and banks of the stream itself, gives an added urgency to the channel stabilization program and also makes possible as an ultimate solution the creation of an essentially sediment-free conveyance channel from Hoover Dam to the lowest diversion.

Yuma Area

The early attempts at bank protection and stabilization works in this basin were intended

to counteract the strong meandering tendencies of the natural river. Early bank protection works by the Bureau in the Yuma area were required to protect the Yuma Levee. The levee was rarely overtopped, but the massive snowmelt floods that occurred each spring often changed the channel alinement until the direct flow of the river was against the levee. When this occurred the levee was undermined and collapsed. These conditions led to the use of heavy quarry-run rock for riprap to protect the levee where it was subject to direct attack by the river. Eventually this riprap was extended to protect the entire levee. The material was not selected as to size but, due to the handling problems involved, stone of about one-half cubic yards was about the practical maximum size.

After the completion of Hoover Dam, no more floods occurred and little if any maintenance work was done on the levee. With the authorization of Morelos Dam, the Mexican diversion structure located in the river below Yuma, the flood hazard in this area was restudied. Although the present flood potential is considerably less than in the natural river, it was still sufficient to require the reconstruction of the Yuma Levee. The levee was reconstructed, in part, to new grade and alinement to meet current conditions. The levee section was increased and heavy riprap placed as indicated. The maximum size of the riprap was limited to one-half cubic yard and was graded down to not less than one-tenth cubic yard. The rock was placed by dumping on the shoulder of the levee and pushing it over the slope with a bulldozer.

The design flood used for the levee rehabilitation was not only smaller in magnitude than pre-Hoover Dam floods but is also not expected to be of as long duration. For this reason, only part of the levee was riprapped. Where riprap was required by the proximity of the river it was placed in blankets either 5 or 10 feet thick measured normal to the embankment slope. The thicker blanket was used only where the alinement of the river threatened to direct the full force of current directly against the levee.

Mohave Valley

About 1944, the change in regime induced by Hoover Dam and Parker Dam resulted in what has been referred to in numerous reports and publications as the "Needles Situation." The backwater from Havasu Lake behind Parker Dam extends into the lower end of Mohave Valley, an area that appears to have been slowly aggrading even under natural conditions. The clear water released from Hoover Dam scoured large quantities of sediment from the streambed and deposited this material in the lower end of Mohave Valley. The buildup of sediment was so rapid that by 1942 the channel was lost and the river flowed through a series of sloughs. The rise in water surface accompanying this aggradation flooded part of the city of Needles, Calif., and threatened the main line and yards of the Santa Fe Railroad.

As a permanent solution to this problem, the Bureau obtained a dredge and in 1949 began work to reopen a channel from Needles to Topock. It was realized that to prevent continued excessive deposition in this channel the river would have to be denied access to the unlimited source of sediment in the upper valley. To achieve this objective it would be necessary to dredge a channel 32 miles in length through the valley and protect the banks of the channel from erosion.

The rectified channel was designed to carry an average flow of 15,000 c.f.s., and levees were provided to contain floodflows varying from 50,000 c.f.s. at the upper reach to 75,000 c.f.s. at the lower reach of the channel.

Bed stability of the rectified channel could not be achieved directly because of the excessive natural grade through the valley. In the design of the channel it was advantageous to adopt a sinuous alinement to reduce the gradient as much as possible as well as for other purposes. By designing to the minimum practicable grade and by confining and deepening the channel, it was possible to obtain conditions under which the bed of the channel can eventually adjust to stability without having to move excessive amounts of bottom material.

Because of the emergency conditions at Needles, the first concern was to open a free-flowing channel through the deposition area. As a result the dredge was put to work to open a pilot channel just wide enough to capture the flow of the river. The river was expected to widen this channel by bank erosion until the design width was reached. Bank stabilization would then be provided as needed to prevent the formation of overwide sections.

Various methods of bank stabilization were tried with something less than complete success. One of these early methods that was tried was a permeable structure designed to cause deposition of the river's sediments in desired areas. These structures were constructed of railroad rails jetted into the bed at 3- to 5-foot intervals supporting 6-inch mesh wire fencing. Initially these structures were reasonably successful, because the river was still carrying a fairly high sediment concentration. However, as the channelization activities began to reduce the sediment load of the river, it was found that these structures no longer performed satisfactorily. It soon became evident that with the velocities of 3 to 6 feet per second encountered

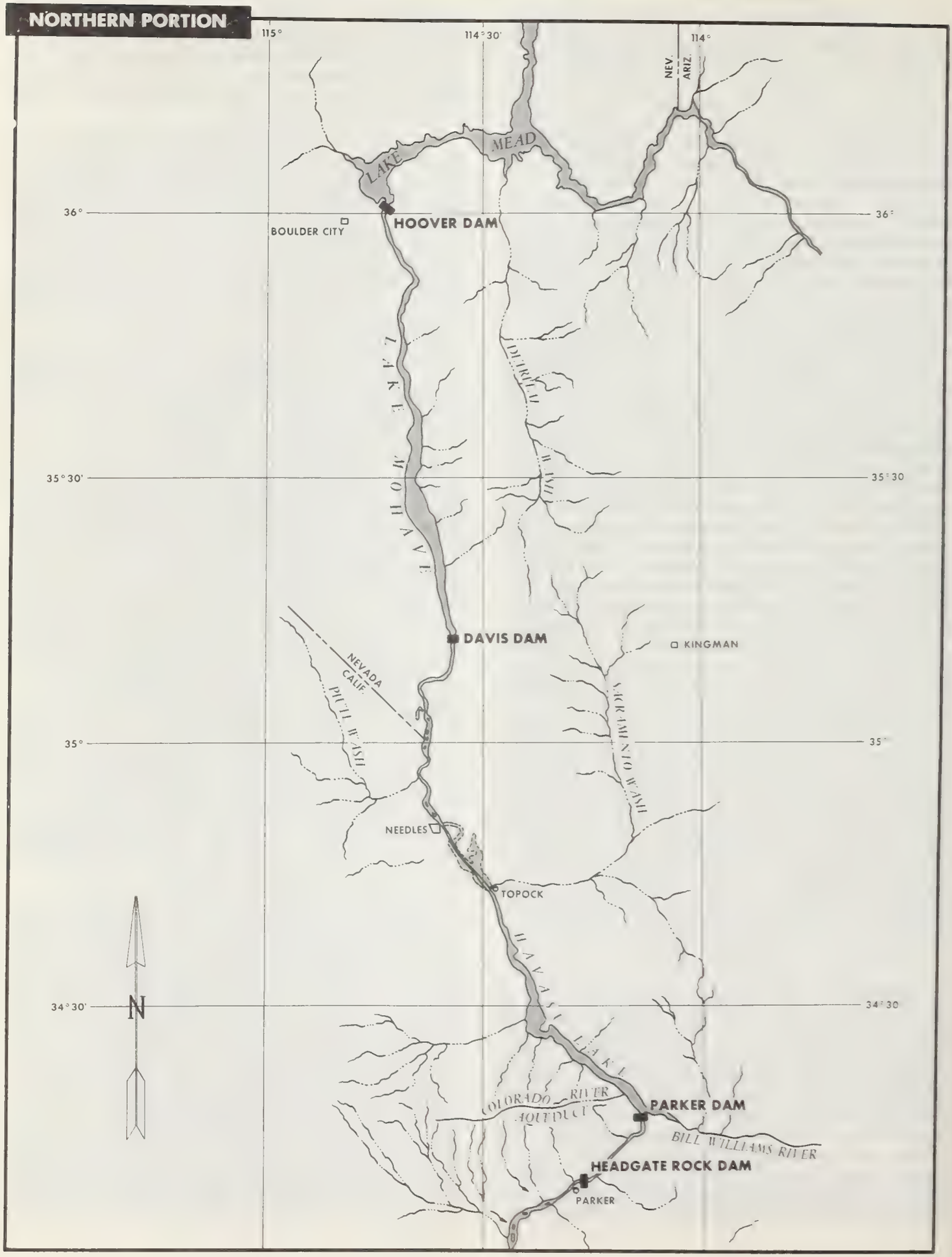
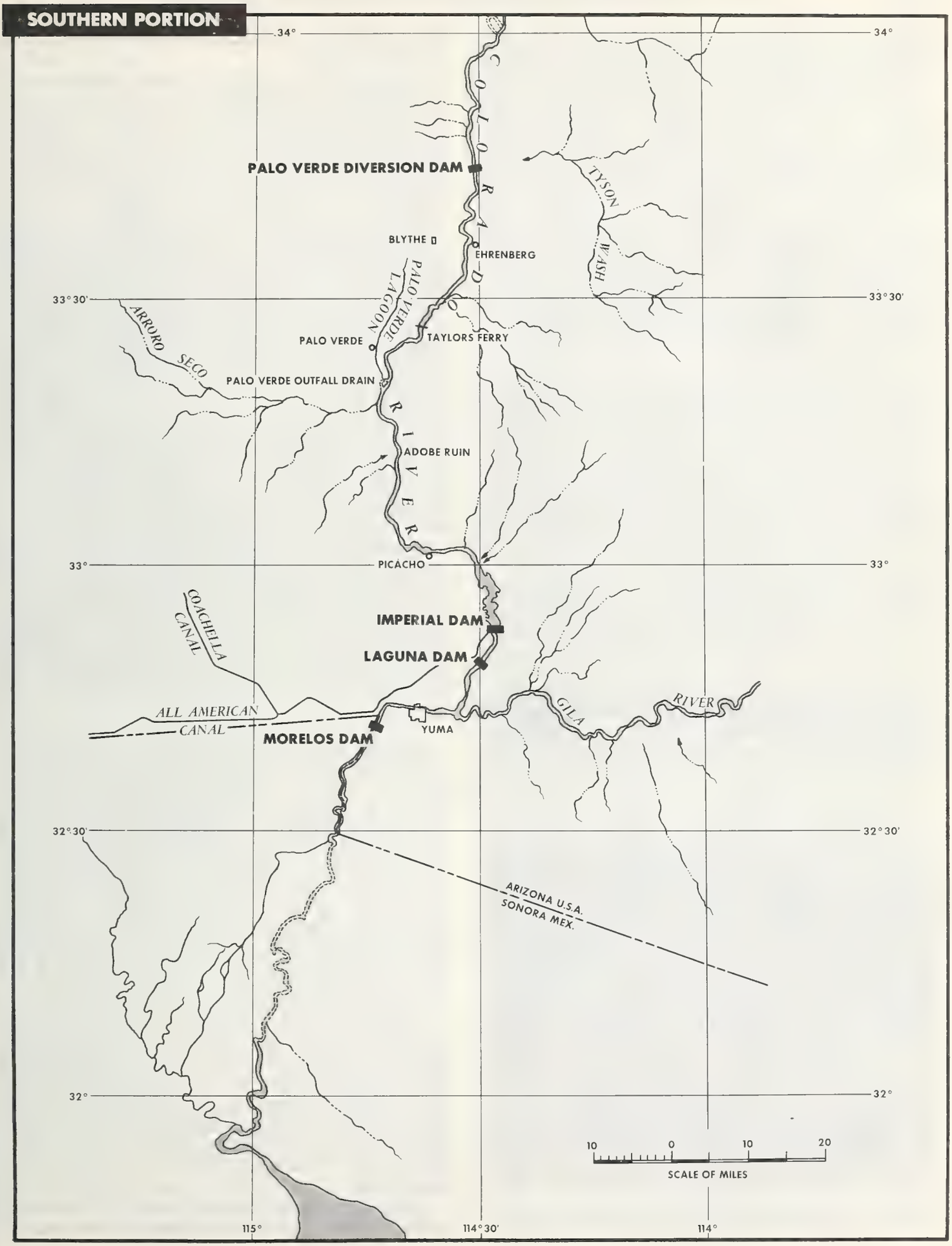


FIGURE 1. — Location map



of Lower Colorado River.

along these banks, the sediment concentration in the flow had to be about 700 parts per million to obtain adequate deposition. When the concentration of sediment dropped to about 300 parts per million or less, the fence itself actually caused some erosion to take place. Jetties formed by Kelner-type jacks were also tried, but with about the same results. In time, all these structures were replaced with rock groins or hard points spaced along the erosion areas. These hard points have been successful in controlling the bank erosion but produce a rough scalloped bank line that may not in all cases be desirable. In addition, such protection does permit some additional bank cutting, which may produce an objectionable amount of sediment in some circumstances.

In later channelization work in the Mohave Valley, a different method of bank stabilization was used that proved highly succ ssful. Because the channel above Needles was to consist of a series of curves, each with a radius of 10,000 feet, and the channel would be dredged to nearly full width, it was decided to place riprap on the concave sides of all bends. This would have resulted in stabilized banks amounting to about one-half of the total length of bank.

Several methods of placing riprap were tried before the final method was adopted. Because the bank material was almost entirely noncohesive sand and silt that was easily erodible, it was possible to place the riprap material on the ground in the form of a windrow and let the river cut the bank away until the windrow was reached (fig. 2). As the river undercut the riprap windrow, the material began to drop down the cutbank and gradually brought the cutting action to a halt.

FIGURE 2. — Riprap material on the ground in the form of a windrow.

Because of the rather large fluctuation of flow in this channel, from about 3,000 to 24,000 c.f.s., and the erodibility of the bank material, it was found that the plan to riprap only the concave bank would not achieve complete stability. As a result, it was necessary in practically all cases to stabilize both banks.

In practice, the windrows were built in advance of the dredge cut on the desired final bank line. The material used for riprap was quarry-run stone with a maximum size of one-fourth cubic yard and with the fines limited to approximately 5 percent. This material was applied at the average rate of about 1½ to 2 cubic yards per linear foot of bank, depending upon the height of the bank. The dredge then opened the channel between the windrows to nearly full width. The material between the dredge cut and the windrow was allowed to erode away and the riprap stabilized the channel bank at the desired location. The final bank line resulting from this procedure presents a neat appearance and has good flow characteristics.

Experience has shown this method of bank protection to be quite successful. Several years have elapsed since this material was placed, and replacement or addition to the original riprap has rarely been necessary. It can be expected that some future maintenance will be required, but judging from past performance this will not require the replacement of a large amount of rock.

Palo Verde Valley

The Bureau of Reclamation has recently begun a program of bank stabilization between the Palo Verde Diversion Dam and Taylors Ferry. This division includes about 27 miles of river and is part of the degradation reach below Parker Dam. The present bed of the channel is entrenched from 12 to 14 feet below the prescour bed. The bed seems to have reached a reasonably stable condition. Either degradation or aggradation may still occur, depending upon flow conditions, but these fluctuations are minor and there is no noticeable trend toward continued degradation.

Despite the general stability of the bed, this reach continues to produce sediment at the rate of about 650,000 tons per year. Since the bed shows little average change, it is apparent that most of this sediment is produced from bank scour. Other observations bear this out, since there are at least 65,000 linear feet of bank in this division which show visible signs of active bank scour.

Although these banks usually cut away rather slowly, only a few feet a year, in one

case measurements show that one bank cut back 300 feet in 1 year.

In addition to bank erosion, there are other channel control problems within this division. Alinement is poor in several places with overly sharp curves that encourage active bank erosion. Although the river in the division as a whole has degraded, there have developed short aggrading reaches in which the channel is overwide and badly braided. Unquestionably the problems of misalinement, overwidth channels, and braiding could have been corrected more advantageously by channelization; however, physical and economic considerations made this solution impractical. As a result, the present program was devised not only to stabilize actively eroding banks but also to improve alinement and other irregularities as much as possible.

The plans for each section of bank to be stabilized is developed individually to fit the particular circumstances at that point. To date one contract has been completed to stabilize about 18,000 linear feet of bank. Included are three structures, designated C–2, A–1, and A–2, that, while not entirely typical, at least illustrate the three main types of stabilization structures planned for this division.

The area selected for the start of this program has the most severe bank erosion in the division. The severity of the erosion is caused by a series of very sharp bends associated with the development of a new meander pattern growing from a cutoff of an older meander. These bends have radii of only 2,500 to 3,500 feet, whereas, experience on this river indicates that for reasonable stability, bends should have radii in excess of 5,000 feet.

In view of the excessive curvature of these banks, a rather high rate of application of riprap was selected. The structure designated C–2 is on the California side of the river on the bend where 300 feet of bank cutting was measured in 1 year. The bend has a radius of approximately 2,500 feet. It was decided to stabilize this bend at its present location. The bank was trimmed to a slope of about 1½ to 1, corresponding to the observed bank angle common to this area. During the trimming process, minor points and irregularities were also smoothed but otherwise the bank alinement was not changed. Since the bank material is "sugar sand," a fine cohesionless sand and silt, a 1-foot blanket of pit-run gravel was placed on the bank as bedding for the riprap. The gravel — natural "wash gravel — was not graded but was limited to 4-inch maximum size. Finally, quarry-run stone was placed on the bank at the rate of 5 cubic yards per linear foot. Specifications limited the maximum size of the riprap to one-fourth cubic yard, and rock spalls or gravel inclusions were not to appreciably exceed the amount required to fill the voids. Both the gravel bedding and riprap were placed by dumping at the edge of the bank and pushing the material over with a bulldozer.

In the case of structure A–1, it was decided that erosion should not be completely stopped. The bend in this area was such that it appeared advisable to allow erosion to continue for a time to obtain a better shaped bank line. A desirable alinement was selected, and, where the present bank coincides with this alinement, the bank was treated similarly to that in structure C–1. Where the final bank line will be back of the natural bank, the riprap material was placed in a windrow along the desired bank line. Since the bank is higher and the curvature greater than the banks in Mohave Valley, the amount of rock in the windrow was increased to 4 cubic yards per linear foot. In a way, this type of structure defeats the avowed purpose of sediment abatement in this division. For this reason, such a procedure would only be used when the long-range benefits of improved alinement seem to be worth the temporary disadvantage of a limited amount of continued sediment production.

Structure A–2 represents a different method of achieving both bank stabilization and improved alinement (fig. 3). This bank combined

FIGURE 3. — Embankment for bank protection and alinement.

a sharp bend, radius about 3,500 feet, with an even sharper hook at the lower end. Incidentally, this is a form of bank development that is fairly common in this part of the river. The corrective action that was adopted in this case was to build a new bank in the river itself. The

entire structure was designed to ease the curvature of the bend as much as practicable and to close off most of the downstream hook. The part of the design structure that coincided with the existing bank was treated in the same manner as the other bank structures. The new bank or training structure was constructed of unclassified gravel material to a top width of 20 feet. Although levee-shaped, these structures are not intended to act as levees and are built only to an elevation that will give a reasonable freeboard at a flow equal to the normal maximum release through the Parker powerplant. The river side of the structure was riprapped in the same manner as the regular banks.

As yet, these structures have not been in place long enough to judge their performance. Certainly in time there will be some shifting of the riprap material particularly if as expected the channel deepens adjacent to the structure and undercuts the toe of the riprap (fig. 4).

FIGURE 4. — Riprap being placed by bulldozer.

Future plans in this division call for several thousand feet more of bank stabilization. The planned structures will generally follow the similar procedures of using riprap on the bank, through the use of training structures, and more rarely, windrows to achieve some measure of improvement in alinement. Since the initial structures were placed on extreme bends, the riprap on these structures are greater in thickness than will be required on later structures. The height of the banks in this area caused by the general degradation of the channel will require somewhat more riprap than was used in the Mohave Valley, and it is expected that the minimum application in Palo Verde will be 2½ to 3 cubic yards per linear foot.

Future Work

The pressing need to conserve as much as possible the available water in the Lower Colorado River Basin requires the continued improvement of the conveyance characteristics and the reduction of the sediment load in the river. Channelization has been authorized in the Cibola Valley but has been delayed, pending the settlement of the Arizona-California State boundary. Studies are presently being made for requirements of channel corrective work in the Parker valley and below Imperial Dam. Bank stabilization will be an important feature of the channelization work in all of these areas because of the nature of the alluvium that forms the bed and banks of the stream.

In Cibola Valley (between Taylors Ferry and Adobe Ruin), the channelization will consist of dredging a new channel about 16 miles in length. The new channel encompasses a fairly stable reach of the river in which there is a transition from degrading tendencies at the upper end to aggrading tendencies at the lower end. Despite the general stability of the bed, it will be necessary to protect the banks of the new channel to prevent enlargement of the channel width. The stabilization will be accomplished in a manner similar to that used in the Mohave Valley, except that in this case it is undesirable to permit as much bank material to erode away before the riprap is launched. Consequently, the channel will be dredged to as near completed width as possible; perhaps even to the extent of partially launching the rock.

Continued activities in the Palo Verde area will extend bank protection to more of the active sediment-contributing banks in this area. Although this work will in general follow the procedures presently in use, the requirement for improving river alinement where possible will no doubt require some revisions and additions.

Conclusions

During the long period in which the Bureau of Reclamation has been engaged in bank and channel stabilization activities on the Lower Colorado River, many methods of stabilization have been tried with varying degrees of success. From this experience the most effective type of bank protection for the conditions encountered on this river has been determined to be quarry-run rock riprap. This material is especially suited to the stabilization of new banks created by channelization and raw banks caused by bank erosion.

New and economical procedures have been developed for placing this type of riprap, which results in neat appearance and low maintenance of the bank line. However, the use of these methods, particularly the use of windrows, require that the bank material be noncohesive and readily erodible in order to launch the riprap properly.

DESIGN OF STABLE CHANNELS IN ERODIBLE MATERIALS

[Paper No. 43]

By PAUL D. DOUBT, *head, Design Section, Engineering Division, Soil Conservation Service*

The methods of permissible velocities, regime theory, and tractive force theory have been applied with various degrees of intensity for the design and maintenance of stable channels. The first two methods are empirical while the tractive force theory is analytical. Some empiricism has been used in the application of the tractive force theory that has unfortunately led to the questioning of the validity of this theory. This paper is given with the hope that it will clarify the logical application of these hydraulic principles and reduce the empiricism used in applying the tractive force theory, by the discussion of three premises:

First, the tractive force theory is logically a dynamic theory;

Second, the water surface profile must be computed to evaluate the tractive forces;

Third, the tractive force depends on the actual depth of flow and on the *roughness coefficient* of the *earth* materials in the wetted perimeter.

The design of an earth channel consists of the following three facets: (1) capacity, (2) stability, (3) economical proportions of the channel and its associated structures.

Hydrological or delivery studies determine the maximum discharge for a channel. This maximum discharge is the design discharge used for determining the size of the channel. A channel is first designed for size to satisfy the capacity requirement. The second phase of design checks the channel for stability and, if necessary, alters the design. The choice of the economical proportions for the channel and its associated structures is coupled with the designs for capacity and stability. Thus, the design of a stabilized channel often requires trial solutions.

In this presentation only the facet of stability design is considered. A channel is considered stable if, for all discharges equal to or less than the design discharge along with their associated sediment loads, the channel banks will not erode or silt and its beds will not degrade or aggrade beyond the designed limits.

Channels are classified into two major groups according to the functions for which they are designed. The procedure for determining the stability in both groups is the same.

One of these major groups consists of channels designed to convey only clear water. The assumption that there is no aggradation of channel bed or sides is used in the design of this group of channels. Thus, no encroachment on channel capacity occurs. The channel is designed to prevent degradation of channel bed and sides.

The other group of channels is designed to convey clear water and to convey the sediment-water complex. These channels are designed to meet their capacity requirements for the maximum aggraded condition. They are designed to meet their stability requirements for the conditions of maximum degradation and clear water flow. These channels usually require stabilizing structures.

Understanding the ways that a grain of sediment is transported facilitates the establishment of the various laws of sediment movement. Sediment is transported by water in three ways. One of these ways is the combination of the other two. The behavior of a particle of sediment can be observed in a model flume having a noncohesive sand and gravel bed. As the velocity of flow increases from zero, the pebble or grain begins to vibrate or oscillate in its rest position. Transportation of a particle takes place in the following sequence:

1. Increasing the velocity of the water causes the pebble to be dragged or rolled from its rest position along the bed in *traction.*
2. A greater velocity causes the grain to jump or skip along the bed. It is called movement in *saltation.*
3. A still greater velocity causes the grain to be carried in *suspension.*

The particles carried in saltation are transported alternately in traction and suspension.

The sediment load is often divided into two groups according to the way sediment is transported past a section.

1. The *bedload* is the rate of sediment transportation by traction.
2. The *suspended load* is the rate of sediment transportation by suspension.

Since it is difficult to measure the rate of the sediment transportation by saltation, such sediment is included in either suspended load or bedload.

The ratio of bedload to suspended load varies widely at different locations and for different stages. The bedload is usually a small percentage of the sediment load, but it is as important as the suspended load. It is the bedload that is chiefly deposited in shoals and on the beds of deltas and alluvial fans at the mouths of rivers.

Tractive force is the force created by the movement of the water over the wetted perim-

eter of the channel and causes sediment movement in traction. It acts in the direction of flow. This force acts over the area of the earth-water interface rather than on the individual particles.

An upward force component created by the motion of a complex current causes transport in suspension.

The design of a stable channel requires the determination of the location of the *evaluative* sections for each kind of material in which the evaluation is made of (1) the maximum actual tractive force for the bed and the sides; and (2) allowable tractive force for each kind of material in the bed and the sides.

The location of evaluative sections for each kind of materials according to their physical properties is determined from the water surface profile. The determination of the location and definition of evaluative sections will be discussed later.

The value of the allowable tractive force (safety factor = 1) is the maximum tractive force that the materials can safely withstand before erosion occurs. In designs the safety factor should be greater than 1.

Determination of Tractive Forces

The hypotheses that sediment movement of sands and gravels in traction is caused by the frictional forces at the boundary is well substantiated by experimentation and observations.

Manning's roughness-coefficient n_t is used to evaluate the frictional force of the earth material in the wetted perimeter of the channel. The relation suggested by Lane [1] is

$$n_t = \frac{D_{75}^{1/6}}{39} \qquad (1)$$

where D_{75} = the size (inches) of the earth materials in the wetted perimeter of which 75 percent (by weight) is smaller.

Manning's roughness-coefficient n_t must not be confused with Manning's coefficient n. The coefficient n is used to evaluate energy losses caused by friction, bends, vegetation, and eddies in a channel conveying water. It is used to determine the water surface profile and is never less than Manning's roughness-coefficient n_t.

The evaluation of the total tractive force acting on the peripheral materials of the channel is a dynamic problem and not a static problem. Thus, tractive force is evaluated by determining the frictional force that is caused by roughness of the earth materials in the wetted perimeter and that produces the energy loss at the earth-water interface. The energy loss at the interface is easily evaluated, since the rate of energy loss caused by the earth materials in the wetted perimeter is known to be

$$s_t = \frac{n_t^2 Q^2}{(1.486)^2 a^2 r^{4/3}} \qquad (2)$$

where

s_t = rate of energy loss at the interface of the earth material and the flowing body of water, foot-pounds per pound of water per foot length of channel;
Q = actual discharge, cubic feet per second;
a = actual flow area, square feet;
r = actual hydraulic radius, feet;
n_t = Manning's roughness-coefficient for the earth material in the wetted perimeter.

The weight of the body of water that flows past a section in the time interval t is $\gamma_w Q t$ pounds where $\gamma_w = 62.4$ = unit weight of water, pounds per cubic foot.

t = time interval, seconds.

The total work done W is the energy loss at the interface. It is the product of s_t and the weight of the body of water, or

$$W = s_t \gamma_w Q t \qquad (3)$$

where W = work done, foot-pounds per foot length of channel.

The total frictional force F that produces this energy loss is obtained from the relation

$$W = FD = F\frac{Q}{a}t \qquad (4)$$

where F = the total frictional force, pounds per foot length of channel.

$D = \frac{Q}{a}t$ = the distance the body of water is moved in the time t, i.e., the distance the force F acts, feet.

Equate equations 3 and 4 and obtain

$$F\frac{Q}{a}t = s_t \gamma_w Q t \quad \text{or}$$

$$F = \gamma_w s_t a \qquad (5)$$

The average unit frictional force on the wetted perimeter is obtained by dividing the total frictional force by the wetted perimeter p or

$$f = \gamma_w s_t r \qquad (6)$$

The unit frictional forces vary in magnitude along the wetted perimeter (fig. 1). They are equal to the unit tractive forces but are oppositely directed.

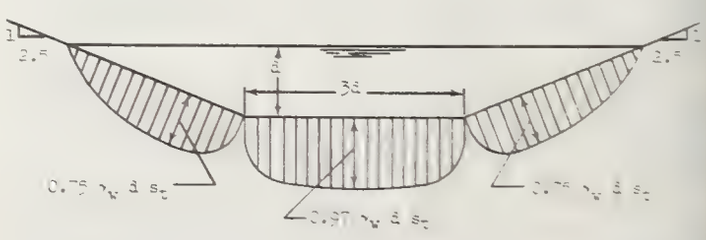


FIGURE 1. — Distribution of actual tractive forces in a trapezoidal channel.

[1] LANE, EMORY W. DESIGN OF STABLE CHANNELS. Amer. Soc. Civil Engin. Trans. 120. 1955.

The maximum tractive force in a trapezoidal section is generally obtained by relating it to the tractive force of an infinitely wide channel. Certain assumptions along with the relation given in equation 6 are used to obtain the tractive force of an infinitely wide channel.

To evaluate tractive forces in an infinitely wide channel of homogeneous earth material, assumptions are made that all energy losses (1) occur on the bed, and (2) are distributed uniformly over its bed.

The first assumption is reasonable for subcritical flow. The second assumption is applicable because the bed of an infinitely wide channel has a uniform distribution of tractive forces. The unit discharge q in an infinitely wide channel has a cross sectional flow area equal to the depth of flow d. Equation 6 becomes

$$f=T_\infty=\gamma_w\, d\, s_t \qquad (7)$$

where

T_∞=tractive force on the bed of an infinitely wide channel, pounds per square foot;

d=actual depth of flow, feet;

s_t=rate of friction loss resulting from flow over the bed materials.

The relations of the maximum actual tractive force on the bed and sides of a trapezoidal section to the tractive force on the bed of an infinitely wide section are based on experimental data and analysis.[2]

The maximum actual tractive force is the maximum tractive force for the evaluative section. The locations of evaluative sections are determined for each earth material in the channel bed and sides. For prismatic channels, the evaluative section is the section having the smallest depth of flow. This occurs at the farthest downstream section in a reach of accelerated flows and the farthest upstream section in a reach of retarded flow. In general, the evaluative section is that section having the largest value of $\frac{d}{K^2_d}=\frac{d\, n_t^2}{(1.486)^2\, a^2\, r^{4/3}}$.

The location of the evaluative section depends on the hydraulic characteristics of the channel and water surface profile computations are required.

The parameter d is evaluated by water surface profile determinations. This value is obtained by use of Manning's coefficient n, which is frequently greater than Manning's roughness-coefficient n_t corresponding to the earth materials in the wetted perimeter of the channel.

The evaluation of s_t at a section in a reach is

$$s_t=\frac{n_t^2\, Q^2}{(1.486)^2\, a^2\, r^{4/3}}$$

The relation of the maximum actual tractive force in natural channels to infinitely wide channel can often be estimated by approximating the actual section with a trapezoidal section.

The actual depth of flow d and the value of s_t associated with the natural channel are used to determine the tractive force T_∞ in the corresponding infinitely wide channel

$$T_\infty=\gamma_w\, d\, s_t \qquad (7)$$

Values of maximum tractive forces for channels with curves are based on judgment coupled with very limited experimental data.[3,4] Channel curves cause the maximum tractive force to increase. The maximum tractive forces, in a channel with a single curve, occur on the inside bank at the upstream end of the curve and near the outer bank downstream from the curve. Compound curves further increase the maximum tractive forces.

Allowable Tractive Force T_{ab}

For noncohesive materials [2] of gradation D_{75},

$$T_{ab}=0.4\, D_{75} \ldots . 0.25 \leqq D_{75} \leqq 5.0 \qquad (8)$$

The allowable tractive force for channel sides T_{as} is less than that of the same materials in the bed of the channel, because the gravity force aids the tractive force in moving the materials. The allowable tractive force for channel sides composed of noncohesive granular materials,[2] on considering the effect of the side slope z and the angle of repose with the horizontal Φ_R, is

$$T_{as}=0.4\, K\, D_{75} \ldots . 0.25 \leqq D_{75} \leqq 5.0$$

where

$$K=\sqrt{\frac{z^2-\cot^2\Phi_R}{1+z^2}} \qquad (9)$$

Thus, the ratio of T_{as}/T_{ab} is K.

When the unit weight γ_s of the constituents of the material having a grain size larger than D_{75} is significantly different from 160 lb./ft.^{3}, the allowable tractive forces T_{ab} and T_{as} are to be multiplied by the factor Γ

$$\Gamma=\frac{\gamma_s-\gamma_w}{97.6} \qquad (10)$$

where γ_s=unit weight of the constituents of material with a grain size greater than D_{75}, lb./ft.^{3}

γ_w=62.4=unit weight of water, pounds per cubic foot.

The evaluation of allowable tractive forces

[2] Bureau of Reclamation. Progress report on results of studies on design of stable channels. Hydraul. Lab. Rpt. HYD-352, 36 pp., illus. 1952. Denver, Colo.

[3] See footnote 1.

[4] Nece, R. E., Givler, C. A., and Drinker, P. A. Measurement of boundary shear stress in an open channel curve with a surface pitot tube. Mass. Inst. Technol. Tech. Note 6. 1959.

for cohesive[5 6] materials has not been as successful as the evaluation of the allowable tractive forces for sands and gravels. There is some doubt regarding the parameters required to define the allowable tractive force of cohesive soils. If the tractive force principle is applicable, it appears that the plasticity index, void ratio, percent clay, along with other possible parameters, may enter into the relation. The time required for degradation of highly plastic soils is indeterminable, because of the scarcity of reliable information.

Existing natural channels show that, as the plasticity index for materials in the channel sides is increased, the allowable steepness of channel sides is increased, provided the channel bed is not degrading.

The allowable tractive force for soils within the range of plasticity index $I_p < 10.0$ and $D_{75} < 0.25$ are difficult to determine. Materials of this region are extremely vulnerable to erosive forces. Channel banks or beds composed of this type of materials will usually require protection. Their design without revetment, lining, or structures is generally impossible.

[5] DUNN, I. S. TRACTIVE RESISTANCE OF COHESIVE CHANNELS. Jour. Soil Mech. and Foundations Paper 2062. 1959.

[6] SMERDON, E. T., and BEASLEY, R. P. CRITICAL TRACTIVE FORCES IN COHESIVE SOILS. Agr. Engin. 42, 1961.

CRITERIA AND METHODS FOR USE OF CHECK DAMS IN STABILIZING CHANNEL BANKS AND BEDS

[Paper No. 44]

By W. R. FERRELL and W. R. BARR, *engineers, Los Angeles County Flood Control District*

Introduction

Erosion of our mountain masses is a very slow process and goes on practically unnoticed in many parts of the United States. However, in regions where there is a high demand for new lands suitable for development and other use by man, the process becomes of prime concern. The importance of stream erosion is amplified by the fact that the areas most directly affected, such as the alluvial fans at canyon mouths, provide not only the room for expansion but also actually are considered as premium building sites.

Although the effects of mountain erosion are now becoming of greater concern in this country, particularly in southern California, such erosion has been recognized and dealt with in the Alpine regions of Europe for centuries. In Europe, many applications of channel stabilization principles were developed and placed into practice with strong reliance on hand labor and the use of on-site materials. In 1954 the Los Angeles County Flood Control District and the U.S. Forest Service in Los Angeles County undertook what probably was the most concerted effort ever made in this country to determine the most effective and efficient means of reducing mountain channel erosion, using modern materials and techniques. A comprehensive report of the findings of this study, which included a review of preceding practices in Europe and the United States, was published by the Flood Control District in 1959 under the title "Report on Debris Reduction Studies for Mountain Watersheds." This study was greatly facilitated by research work conducted in this area by the U.S. Forest Service in the 1940's under the Los Angeles River Watershed "Memorandum of Understanding." The plan set forth in the report provides for a $20,000,000 stabilization program that is currently under way. As the work progresses, the design and construction techniques established in the report are continually updated as experience dictates.

Evidence of Need for Stabilization

In various parts of the United States the need for bank and bed stabilization may vary, depending upon physiography, land use, and economic justification. In the semiarid Southwest, and especially the coastal plains and valleys of southern California, alluvial fans below the canyon mouths have aggraded for centuries, fed by sheet erosion and channel downcutting with its accompanying bank sloughing within the mountains. The complete occupancy of these fans below the smaller frontal canyons has necessitated a comprehensive system of debris dams, debris basins, and lined channels to desilt mountain streams and convey the desilted flows to spreading basins or to the ocean.

Trapping silt and debris in the basin is only part of the job. These erosion products must then be removed and disposed of, and along with the excavation and hauling costs comes the problem and expense of locating and purchasing disposal sites. The latter must also be located near the canyon mouths to reduce haul distances.

In the larger canyons, reservoirs built and maintained for the purpose of flood protection and water conservation become silted to the point where storage necessary for flood regulation or worthwhile conservation activities becomes inadequate. Desilting is an expensive and

frustrating operation that involves sluicing and subsequent removal from downstream works or mechanical excavation and disposal similar to that of a debris basin. It can readily be seen that competition for available land in the foothill zone is entered into by both public agencies and private citizens.

Primarily as a result of a medium-sized storm in the 1961–62 storm season, the Los Angeles County Flood Control District excavated 1,235,600 cubic yards of debris from its debris basins at a cost of $1,044,600, or about $0.85 per yard, which does not include various extras such as disposal site costs. In actuality the total costs are currently running close to $1.50 per yard. The great increase in cost is manifest when compared to the 1938 cost of $0.33 per yard. Figure 1 shows Puddingstone diversion dam before and after 328,000 cubic yards of debris had been removed. During the same storm the storage loss in 11 of the District's major flood-regulating and water conservation reservoirs was computed to be 1,720 acre-feet (2,768,500 cubic yards). These figures would be increased manyfold for a capital storm. The need for stabilization to reduce erosion in southern California, therefore, is based primarily on economic considerations.

Design of Stabilization System

Through experience and experimentation a procedure has been developed that reduces to a minimum the time required to prepare plans for a stabilization system. The planning consists of three basic phases: field engineering, office engineering, and economic analysis. The field engineering phase is very important in the development of a stabilization system because every effort must be made to utilize desirable features provided by nature. The engineer is guided by two basic rules in the utilization of these natural features; these are (1) the entire channel reach being considered must be stabilized, either by existing natural conditions such as exposed bedrock, natural dams, etc., or by the stabilization system proposed; and (2) the structures should be so located that the spillway height above natural streambed does not exceed 17 feet nor is less than 10 feet. This rule is based on economic considerations of structure design, as will be discussed later. The importance of keeping the proper balance between site conditions, continuous stabilization, and desirable structure height cannot be overemphasized. The distance upstream that stabilization can economically be justified must be estimated for the initial layout. In southern California it has been found that this economic limit is reached when the channel gradient approaches 20 percent; however, in special cases a 25 percent limit may be permissible. Other considerations may modify the termination point, such as access road costs.

The definition of a stabilized reach as used above is that state of channel regime at which all downcutting is halted and bank erosion is reduced to an absolute minimum. To accomplish this requires that the successive stabilization structures be so placed that the debris impounded by one will create an artificial gradient that terminates at the toe of the next structure upstream (fig. 2). This gradient has been found to be approximately 0.7 of the natural gradient and is the figure used for design and layout purposes. Figure 2 is a cutaway sketch showing this stabilized condition.

Experience has shown that accurate topographic maps with a 2-foot contour interval, including all terrain lying within 30 feet vertically of the channel bottom, are highly desirable for use in connection with the field layout, and a precise profile along the streamline itself

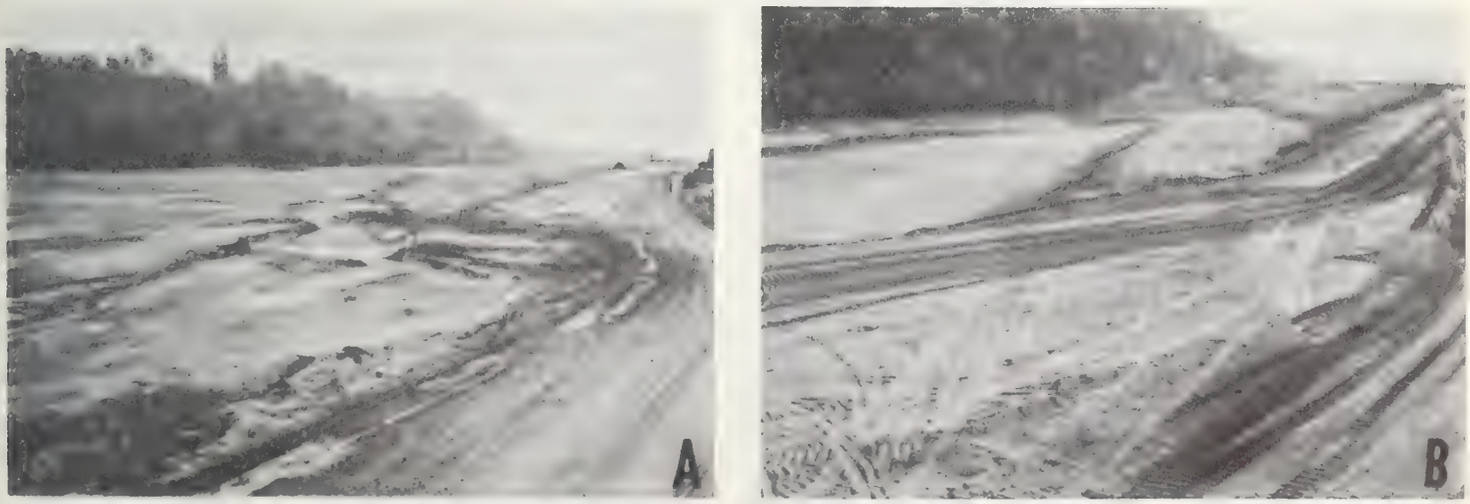


FIGURE 1. — Puddingstone diversion dam after 1961-62 storm season (*A*) and after debris was removed (*B*).

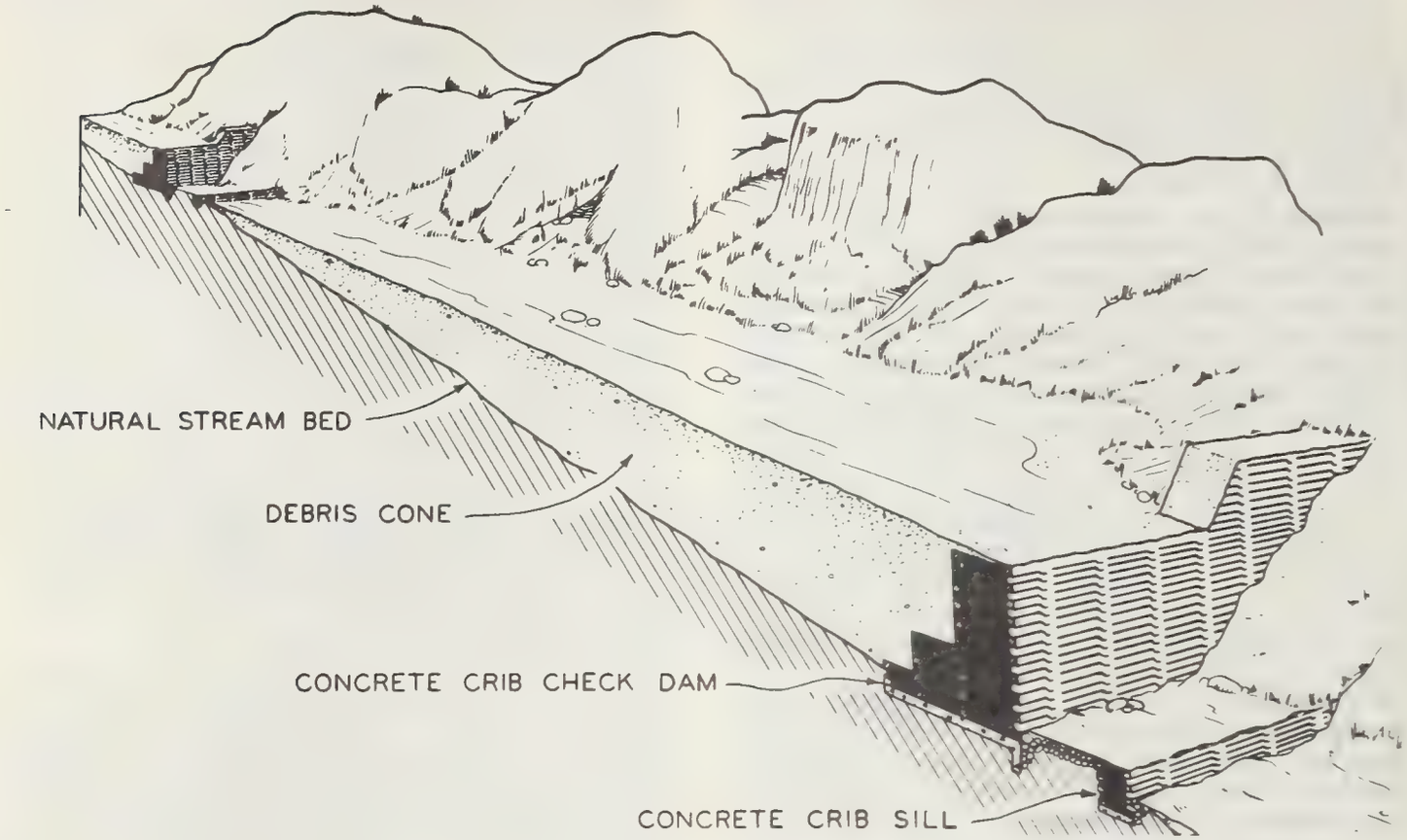


FIGURE 2. — Cutaway view of channel stabilization system.

is necessary. The use of topographic maps based on aerial photographs as aids in planning has shown poor results, especially where vegetation covers the canyon bottom at the time of the photograph. Small falls of 5 to 15 feet in height, critical in the design of a channel stabilization system, are usually unaccounted for in this process. After tentative field location of all structures has been completed on the basis of the foregoing, the office engineering phase commences. Accurate topographic maps enable the office engineer to locate the best site for the structures, to establish required rights of way, and to determine project benefits.

A detailed study of various types of construction materials that have been used extensively for stabilization structures, and some new ones yet untried, was made as a part of the previously mentioned comprehensive study published in 1959. On the basis of cost, required height, ease of construction, availability of backfill material, access possibilities, and maintenance, certain types of construction materials were found to be particularly well suited for specific situations. The materials, in order of general preference, are concrete crib, metal bin, rubble masonry, and such miscellaneous materials as corrugated metal arch, timber crib, and soil cement. Study of the site, access, and size of watershed will usually indicate the need for a structure of either concrete crib or metal bin (fig. 3). Concrete crib is used at all locations where access is not a prime consideration, as it offers several additional and desirable features in the form of structural stability, longevity, and construction flexibility. The metal bin type is often used in regions where access is difficult and expensive, and airborne operations are indicated.

Important features in the design of concrete crib structures include the basic concept of a stable, yet most economical unit possible for a given height of structure and design depth of flow over the spillway. To insure safety against the most common cause of check dam failure that occurred in the past, namely, undercutting at the toe and abutments, all foundations not on bedrock are designed as continuous reinforced concrete spread footings. A cutoff wall is provided at the downstream edge to protect against undercutting and to eliminate foundation piping by insuring a hydraulic gradient of less than unity at all points under the structure.

Additional safety against erosion at the toe from the pounding of desilted flows passing over the spillway is provided in the form of a transverse sill placed downstream at a distance equal to the spillway height plus 5 feet, with the in-

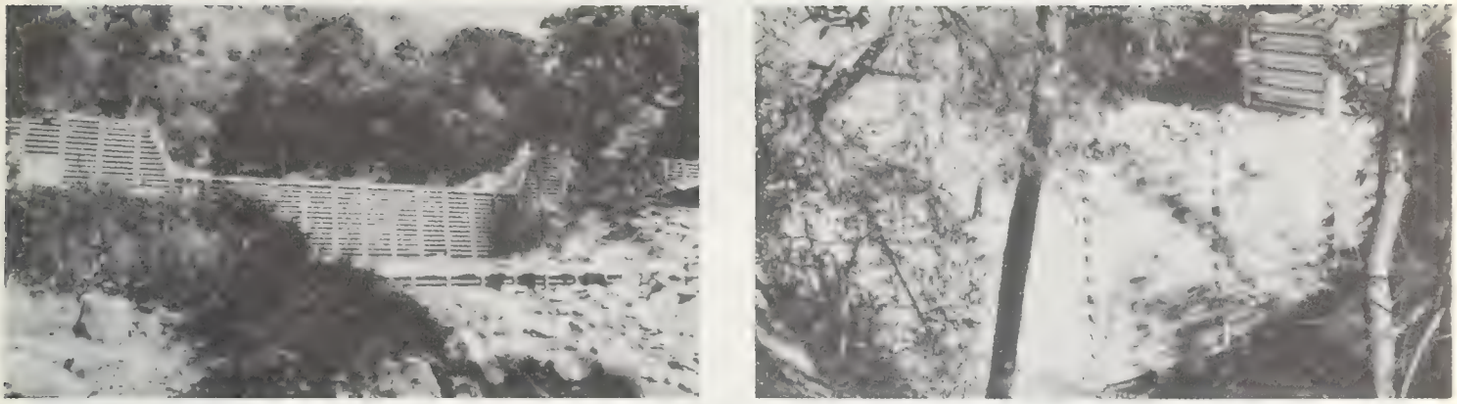


FIGURE 3. — Typical stabilization structures: Left, concrete crib; and right, metal bin.

terspace filled with boulders. These sills become largely inoperative once the debris cone from the structure downstream reaches design elevation. Spillways are gunited to provide a tough wearing-surface against the abrasive action of sand and gravel, and a gunited low-flow chute is provided down the face of the structure. The abutments are stepped as the canyon sidewalls permit, based on the module of the cribbing (6 feet). Economic analysis of various structure sizes has indicated that the structure height should not be less than 10 feet from streambed to spillway, or foundation costs will comprise too great a part of the total. The maximum height is generally limited at 17 feet by the allowable bearing value of concrete at the "column points" formed by the intersection of crib members where structure loads are transmitted to the foundation. For higher structures these "column point" areas must be increased by the use of double crib members or by special oversized members. This has been found too costly with respect to additional benefits, except in special cases. Figure 4 shows a typical concrete crib structure.

The use of metal bins for stabilization structures has involved the same careful approach, but with extra precautions taken to insure sliding stability (fig. 5). This is necessary, because metal bin structures are placed at existing streambed grade with no possibility of keying action such as is provided by the cutoff wall in the case of the concrete crib. The metal bin type construction is not subject to a rigorous analysis. Therefore, to solve unanswered design questions, a special test structure packed with pressure gages, piezometers, and strain gages was constructed at a typical canyon site in cooperation with the Metal Products Division of Armco Steel Corp. The structure was loaded to the theoretical point of failure and observations of settlement, horizontal movement, and structure stress and strain were made during the loading program. This research has provided assurance that the design criteria developed for metal bin dams will provide the utmost in economy but with no sacrifice in stability.

If the particular area to be stabilized is within a very small headwater area or a small tributary branch to a main channel, such materials as soil cement, timber crib, or corrugated metal arch may be found desirable. Each reach of channel requires analysis in all respects, with the appropriate choice of structure type based on the above criteria. Design manuals, which include nomographs and charts, have been developed to facilitate a full design of each structure. (See fig. 6 for an illustration of rapid design procedure.) After the design is completed, the customary computation of volumes and quantities of materials is made, followed by the cost estimate. Added at this time must be the cost of delivering materials to the site, namely, the provision of access, the cost of which must be distributed appropriately over the structures served. In many watersheds the presence of high falls, narrow canyon passages, or long reaches of stable channel will necessitate a change from concrete crib construction, where access roads are used for delivery of equipment and materials, to metal bin structures which utilize helicopters for this purpose. Taken into account must be the cost of building access roads as opposed to that of airlifting materials to the site. Also, hillside road construction can sometimes create more erosion products than are eliminated by the planned stabilization system.

Right-of-way acquisition is left for consideration at this time because of the several factors that affect the boundaries of lands required. It has been this flood control district's experience that property owners often are willing to cooperate after they realize that the stabilization of channels through their lands will work to their advantage and access roads will in many cases be open to them. Easements for construction

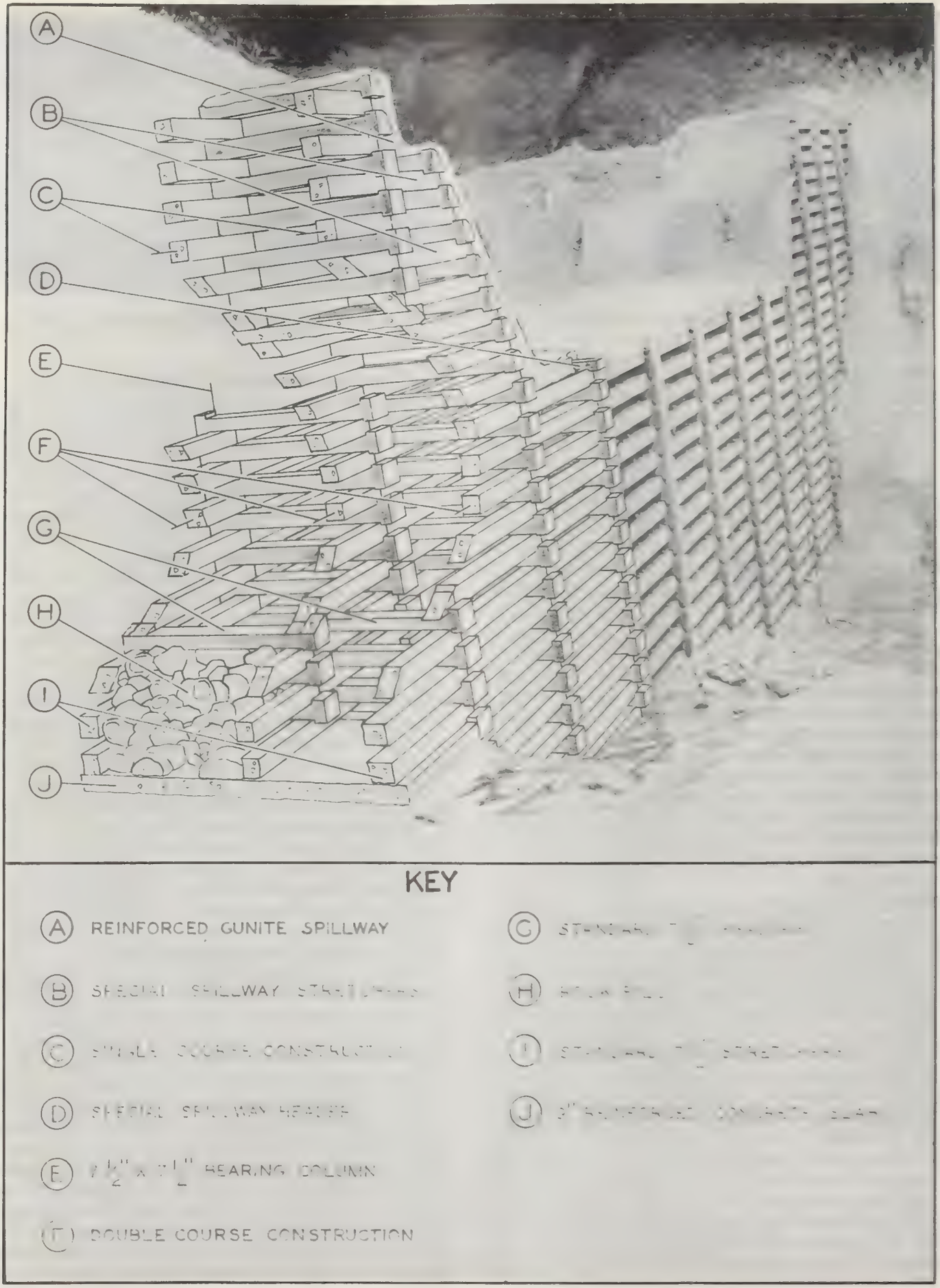
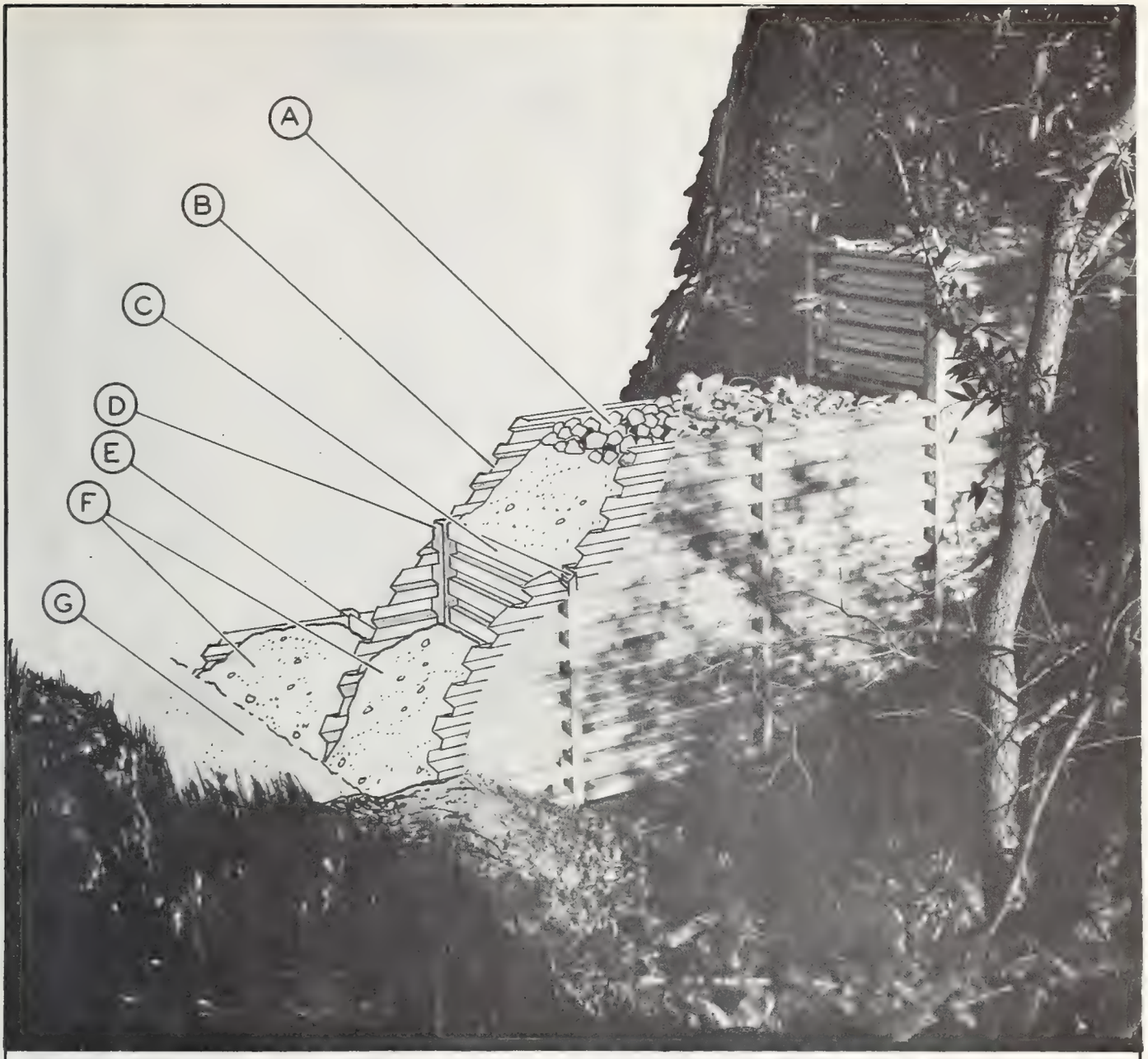


FIGURE 4. — Combination photograph and cutaway drawing of a concrete crib structure designed for the Cooks Canyon project.



KEY

(A) ROCK SPILLWAY SURFACE

(B) STRINGER

(C) SPACER

(D) CONNECTOR

(E) REAR BIN (FOR ADDED STABILITY)

(F) COMPACTED BIN FILL

(G) ORIGINAL STREAM BED

FIGURE 5. — Combination photograph and cutaway drawing of a typical metal bin structure.

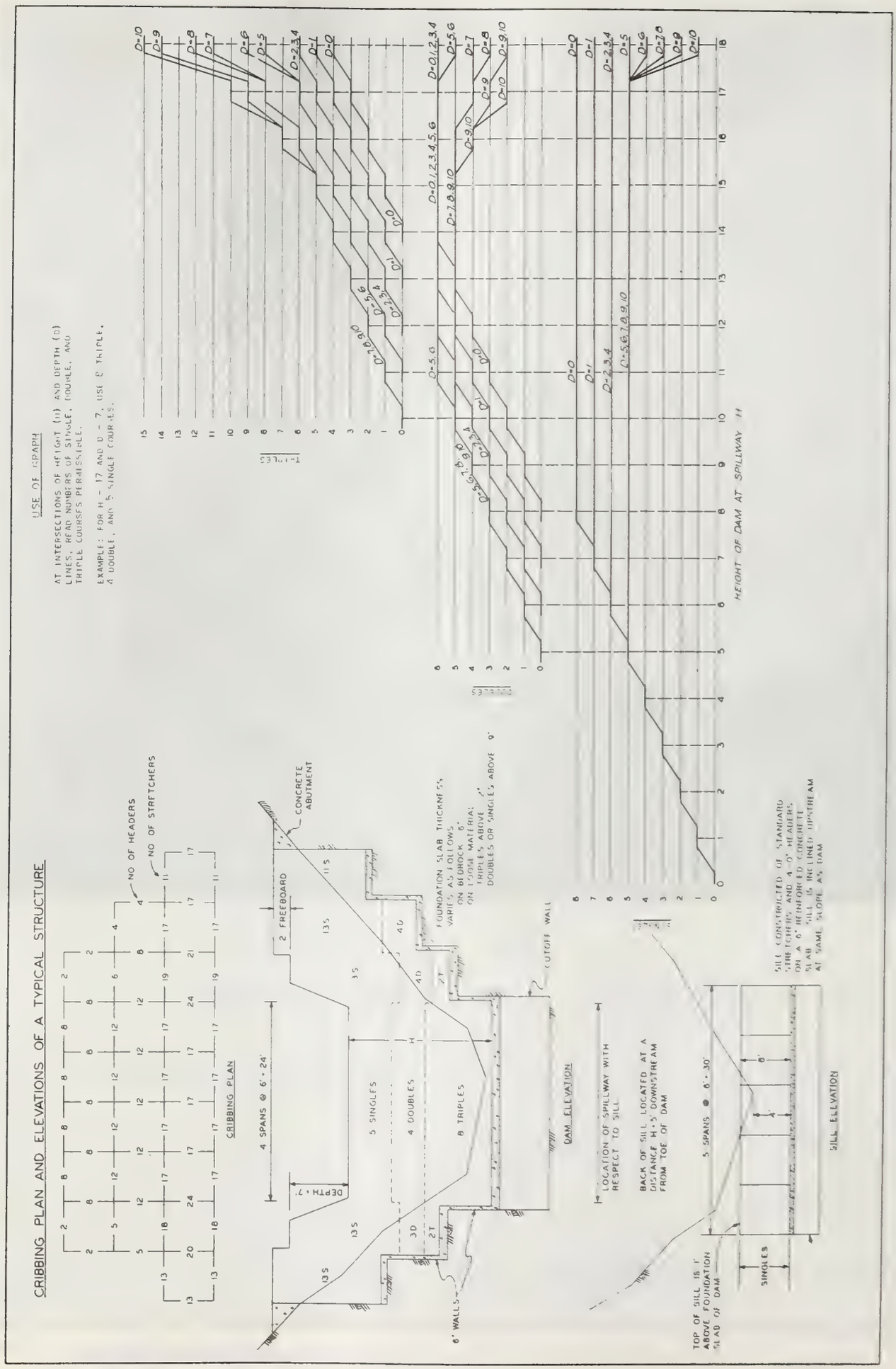

CRIBBING PLAN AND ELEVATIONS OF A TYPICAL STRUCTURE
CRIBBING PLAN
NO OF HEADERS
NO OF STRETCHERS
4 SPANS @ 6' - 24'
2 FREEBOARD
CONCRETE ABUTMENT
DEPTH + 7'
5 SINGLES
4 DOUBLES
3 TRIPLES
6' WALLS
FOUNDATION SLAB THICKNESS VARIES AS FOLLOWS ON BEDROCK 6' ON LOOSE MATERIAL: TRIPLES ABOVE 2' DOUBLES OR SINGLES ABOVE 0'
CUTOFF WALL
DAM ELEVATION
LOCATION OF SPILLWAY WITH RESPECT TO SILL
BACK OF SILL LOCATED AT A DISTANCE H-5' DOWNSTREAM FROM TOE OF DAM
TOP OF SILL 18' ABOVE FOUNDATION SLAB OF DAM
5 SPANS @ 6' - 30'
SILL CONSTRUCTED OF STANDARD STRETCHERS AND 4'-0" HEADERS ON A 6" REINFORCED CONCRETE SLAB. SILL IS INCLINED UPSTREAM AT SAME SLOPE AS DAM
SILL ELEVATION
USE OF GRAPH
AT INTERSECTIONS OF HEIGHT (H) AND DEPTH (D) LINES, READ NUMBERS OF SINGLE, DOUBLE, AND TRIPLE COURSES PERMISSIBLE.
EXAMPLE: FOR H - 17 AND D - 7, USE 2 TRIPLE, 4 DOUBLE, AND 5 SINGLE COURSES.
HEIGHT OF DAM AT SPILLWAY H

FIGURE 6. — Cribbing plan, elevations, and overflow gravity section design for typical stabilization structures.

and operation and maintenance of structures within their lands are often obtained at little or no cost. However, since some expenditures are likely to be incurred in connection with right-of-way, this item must be considered in the total cost estimate. The topographic map of the channel bottom area discussed previously is used to define the limits of right-of-way needed. The general criterion is to acquire easements or fee title if necessary for the channel bottom and the side slopes to a distance of 5 feet above the computed high-water line. In some cases right-of-way must also be obtained for access roads.

The final step in the design phase is the economic analysis. An evaluation of the benefits realized from the installation of a stabilization system are needed for a determination of the benefit-cost ratio. All benefits are not readily apparent, and experience has shown that further research is necessary (and is being conducted) to provide the parameters necessary for their conversion to dollar values. Those benefits that can be converted are:

(1) The value of the new debris storage capacity available above existing streambed behind each of the structures of the system. The debris that will be impounded results mainly from sheet erosion and small side canyon flushing, since channel downcutting and its associated bank sloughing will have been halted.

(2) The net reduction in erosion from the watershed. This is the long-time reduction that is achieved by virtue of the halting of channel downcutting and associated bank erosion. This benefit has been very difficult to evaluate, because a progressive reaction takes place opposite to the process of accelerated erosion; that is, as the channel bottom stabilizes and aggrades, the side-slope scars are able to stabilize at the toe, thus allowing native vegetation to re-establish and eventually heal the scar area completely.

(3) The value of water conservation. Water is conserved as a result of the recurrent filling during storms of the voids within the porous debris deposits behind each structure and their subsequent draining during dry periods. This amounts to a considerable volume of temporary storage for water that otherwise might tax downstream conservation facilities during storm periods and thus be lost for conservation use. Studies have shown the voids available for this temporary storing of water will average around 35 percent of the total volume.

The monetary value of the benefits accruing is ascertained on the basis of (1) unit costs to remove debris from downstream facilities in order that they remain operable; and (2) the cost to the district of purchasing water from outside sources for ground water replenishment. The present figures being used for debris removal and purchased water are $1.50 per cubic yard and $14.00 per acre-foot, respectively.

Among the intangible benefits for which no attempt has been made to arrive at a monetary value are (1) the promotion of vegetation on the stabilized bank slopes that can take place once the halting of side-slope sloughing has been accomplished; and (2) the change in shape and character of the channel bottom, which, as the V-shaped canyons aggrade and become U-shaped, give vegetation an opportunity to establish itself in the channel bottom area. These benefits provide areas for recreational use high in esthetic value.

Once the monetary value of the benefits has been established, the ratio of benefits to cost is determined on the present worth basis, capitalized over a 50-year period. Fifty years is considered to be a conservative period in view of the ruggedness of the materials used. However, experience has proved that an annual maintenance cost will be incurred and therefore must be included in the benefit-cost analysis. A benefit to cost ratio of one or greater is normally considered justification for proceeding with the project. This position is reinforced by the above-discussed intangible benefits that help meet the expanding need for new recreation areas and the protection of existing facilities.

Construction Procedure

In order to make maximum use of both labor and equipment, the construction of stabilization structures must be organized on an assembly line basis. While the structure at one end of a system is in the cleanup stage, site clearing is under way further along the line of progress, and structures in between will be in intermediate stages such as foundation-excavation, foundation-pouring, crib member placement, etc.

Routing over access roads and selection of staging areas should be carefully planned to limit traffic interference with construction. This is especially important since the roads are generally one lane wide, and may often pass over the abutments of partially completed structures.

Experience has shown that the costs of obtaining complete foundation analyses, although required if final structure design drawings are prepared prior to construction, are high and unnecessary. Every effort should be made to work with nature, and this is particularly true at this point where much excavation can be saved by timely decisions in the field during construction as to foundation depth and stepping, with subsequent savings all along the line. To provide this flexibility, it is desirable that a

well-qualified project engineer be on the job, empowered to make such revisions as field conditions indicate.

Once a site has been located on the ground, a system of wires, set across the canyon by a survey crew to mark the positions of the toe and heel of the dam, will greatly facilitate construction. Plumb bobs, marking abutment breaks, the spillway, or other special features, are suspended from these lines, permitting rapid checks during excavation and initial construction. Vertical control is established by temporary bench marks with elevation checks as required.

Equipment should be carefully chosen with respect to the type of structure to be built. When the concrete crib construction is utilized, D–8 or D–9 dozers, often augmented by explosives for site clearing and foundation excavation, are frequently needed. A backhoe is normally used for cutoff walls and minor abutment excavations, whereas a crane of ½- or ¾-yard capacity is necessary for handling heavy crib members and placing concrete.

To conserve labor, forms only for the foundation are specified for most concrete pours, and screeding is done with a specially constructed aluminum rail fitted with handles. Reinforcing steel is bent in prescribed fashion at a staging area before transport to the site.

Fill material for concrete crib structures is specified as clean rock in the downstream bays only, and pit-run material from the channel bottom is used for upstream bays. This procedure is used to reduce settlement and leaching from the downstream bays and also to reduce the total quantity of clean rock required. However, in some canyons the volume of rock needed will still make quarrying necessary, which greatly increases costs and should be kept to a minimum. Figure 7 shows typical construction details, and figure 8 shows three views of construction progress on a typical structure.

Metal bin structures, due to their use in less accessible locations, require relatively small, compact, and often self-contained equipment. Often a good deal of undesirable and expensive hand labor is required at present, but plans to develop highly versatile equipment, transportable by air, are being considered to reduce costs.

Operation and Maintenance

Depending upon the types of structures included in the project, certain repair work will be required periodically, particularly in the vicinity of the spillway. When repairs are made, such special equipment and materials as self-contained hand portable tools and epoxies should be used.

Access routes to the sites, whether trail or road, should be kept passable to assure rapid attention to necessary repairs. Experience has shown that annual inspections are needed after each storm season to assure correction of maintenance problems.

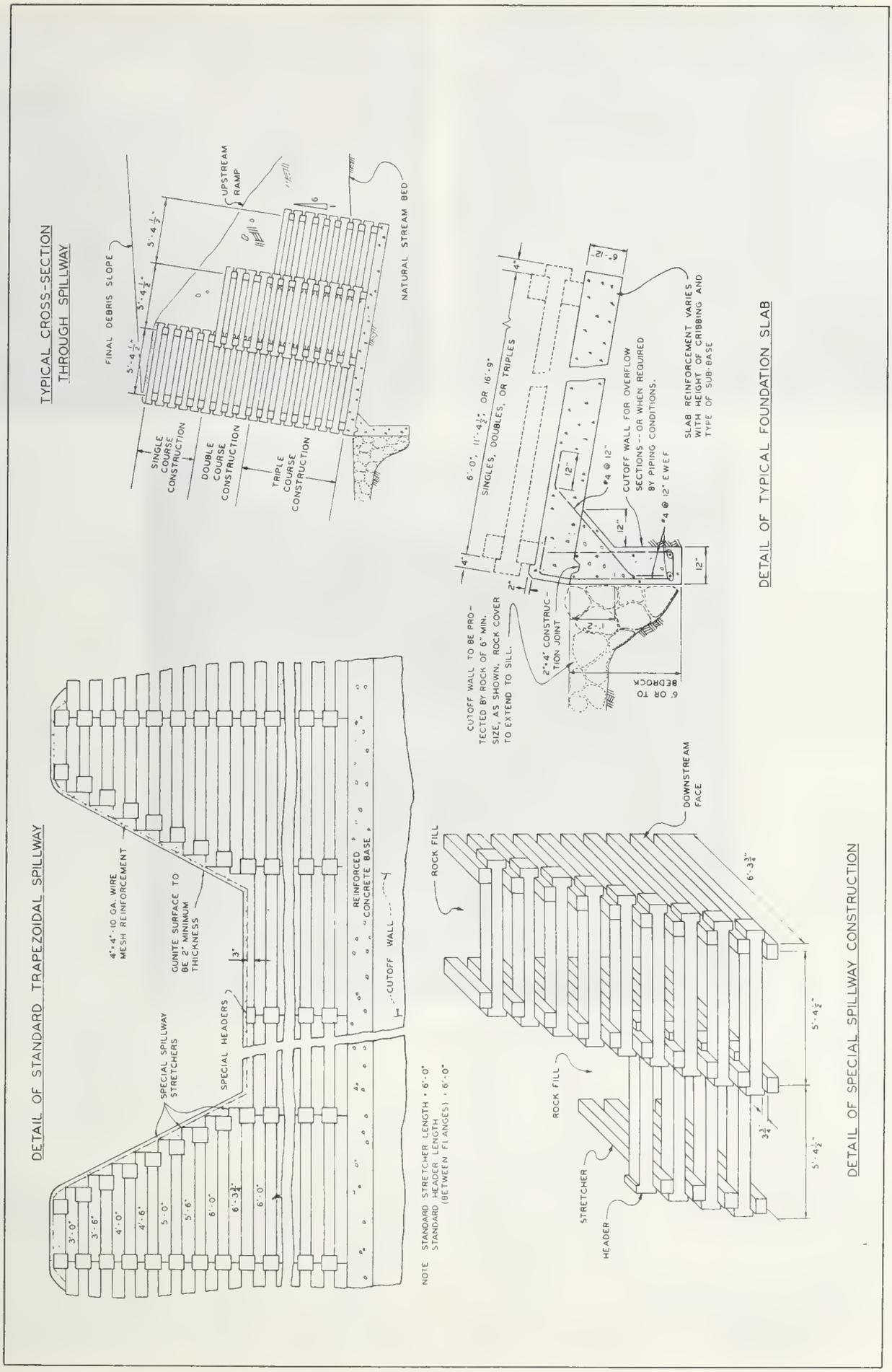

TYPICAL CROSS-SECTION
THROUGH SPILLWAY
FINAL DEBRIS SLOPE
UPSTREAM RAMP
NATURAL STREAM BED
SINGLE COURSE CONSTRUCTION
DOUBLE COURSE CONSTRUCTION
TRIPLE COURSE CONSTRUCTION
DETAIL OF STANDARD TRAPEZOIDAL SPILLWAY
4"-4"-10 GA. WIRE MESH REINFORCEMENT
GUNITE SURFACE TO BE 2" MINIMUM THICKNESS
SPECIAL SPILLWAY STRETCHERS
SPECIAL HEADERS
REINFORCED "CONCRETE BASE"
CUTOFF WALL
NOTE STANDARD STRETCHER LENGTH = 6'-0"
STANDARD HEADER LENGTH (BETWEEN FLANGES) = 6'-0"
ROCK FILL
DOWNSTREAM FACE
STRETCHER
HEADER
DETAIL OF SPECIAL SPILLWAY CONSTRUCTION
CUTOFF WALL TO BE PROTECTED BY ROCK OF 6" MIN. SIZE, AS SHOWN. ROCK COVER TO EXTEND TO SILL.
6'-0", 11'-4½", OR 16'-9" SINGLES, DOUBLES, OR TRIPLES
2"-4" CONSTRUCTION JOINT
BEDROCK
6' OR TO
#4 @ 12"
CUTOFF WALL FOR OVERFLOW SECTIONS -- OR WHEN REQUIRED BY PIPING CONDITIONS.
#4 @ 12" EWEF
SLAB REINFORCEMENT VARIES WITH HEIGHT OF CRIBBING AND TYPE OF SUB-BASE
DETAIL OF TYPICAL FOUNDATION SLAB

FIGURE 7. — Design and details for concrete crib check dam construction.

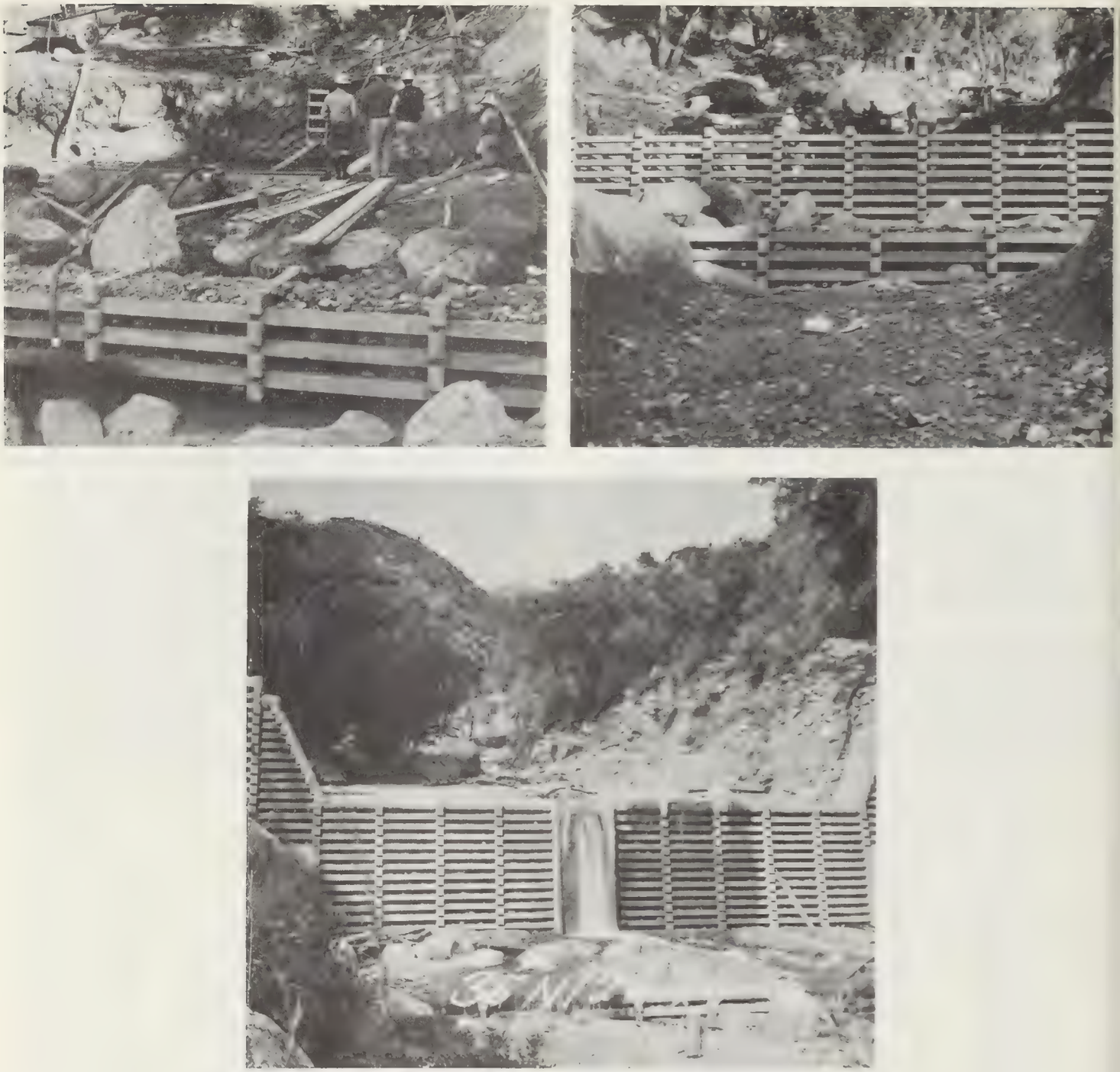


FIGURE 8. — Construction progress on a typical structure.

BANK STABILIZATION OF MISSISSIPPI RIVER BETWEEN THE OHIO AND MISSOURI RIVERS

[Paper No. 45]

By A. J. TIEFENBRUN, *civil engineer, U.S. Army Engineer District, St. Louis*[1]

Synopsis

Regulation, control, and stabilization of channel achieved on an alluvial river are dependent upon the specific demands of navigation and flood control. The earliest plan of improvement in the middle Mississippi consisted of closing dams or solid dikes constructed of brush and stone to confine the flow to a single channel. Experiments with various types led to the adoption of the permeable structure consisting of pile dikes and crib dikes. Although dikes cause deposition and accompanying accretions, in many instances the dike system may induce a heavy attack on the opposite bank downstream. Unless banks are protected, large scale caving will occur. Standard bank protection consists of an underwater part (lumber mattress and stone ballast) and upper bank part (riprap paving). In addition to controlling the trace of the channel at low water, bank protection must be adequate to prevent meandering at high stages. Experimental stone foundation has been used in the repair of revetments where the river attack is mild and the toe of the bank is above or near the low water plane. This type of foundation has proved more stable and less costly than lumber mattress foundation. Sediment from upstream overbank areas and from bank erosion still are factors in regulating river works, but their contribution is diminishing. Indications are that we are now entering a stage in the middle Mississippi where impermeable dikes may be needed. Present methods of river regulation, control, and stabilization have proved effective and at the lowest possible cost. However, new improved methods are constantly being searched for. Improvement over present methods will probably come under the impact of increasing navigation demands and the trend for greater utilization of flood plain areas.

Bank Stabilization

The methods and degree of regulation, control, and stabilization of channel achieved on an alluvial river are dependent upon the specific demands for navigation and flood control during a particular era. The outstanding example of progressive degrees and methods of stabilization for an alluvial river is the Mississippi. Here, the bank stabilization program dates back well over a century. In order to understand the nature of the land through which this river flows, the erosive effect of the river current action on its banks, and the natural changes in the river's regimen, a brief description is given of the more important physical characteristics of its basin.

The Mississippi River drains an area of 1,240,000 square miles and extends from the Appalachian Range on the east to the Rocky Mountains on the west and from the lower St. Lawrence watersheds on the north to the Gulf of Mexico on the south. For general consideration, the Mississippi may be divided into three major reaches or sections — the upper, middle, and lower Mississippi. The river above the mouth of the Missouri is commonly called the upper Mississippi, and from the mouth of the Ohio to the Passes the lower Mississippi, the reach between the two being referred to as the middle Mississippi. The several reaches present an almost endless diversity in topography and geology.

The evolutionary life of the Mississippi River is closely associated in geologic history with the Glacial Age in North America. The Continental Ice Cap in its extreme advances reached as far south as the Ohio River, once considered to be the mouth of the Mississippi. For a time, before the Gorge of Niagara opened to pass the waters of the Great Lakes, drainage of these lakes reached the Mississippi through the Illinois River. In a similar way, drainage in southern Canada poured into the Mississippi valley until the ice had receded to the Hudson Bay and drainage to the north was then possible. As a result of these geological actions and this period of glacial drainage, the bed of the Mississippi River was worn down by the erosive action of a much greater volume of water than that of present day ordinary flow. At present, the river follows a winding course between low banks in a very wide flood plain bordered by high bluffs of sedimentary rock.

My detailed observations and discussion will be confined to that reach of the Mississippi between the Ohio and Missouri Rivers. In the middle Mississippi, the distance between bluffs averages about 5 miles, except in the vicinity of

[1] The author obtained some of his information from papers entitled—

"Navigation Work of the Corps of Engineers on the Middle Mississippi River and Its Relation to Flood Flows," by J. C. DeBolt, U.S. Army Engineer District, St. Louis. 1947.

"Bank Protection on Mississippi and Missouri Rivers," by J. H. Jackson, Corps of Engineers. 1935.

"Protection of Beds and Banks of Inland Waterways, Deep Water Channels and Drainage Canals," by H. D. Vogel, Corps of Engineers. 1940.

Sawyer Bend, opposite the northern boundary of St. Louis, where it attains a maximum width of about 12 miles. As a result of filling of the old rock-bound valley with glacial outwash, the present river, in general, flows in a bed of glacial material with bedrock at a considerable depth below the streambed. In its natural state, the river between high banks was relatively wide and shallow, with the channel being bifurcated by numerous bars and islands with their accompanying chutes and sloughs. At low water, channel depths varied from 3 to 10 feet over widths ranging from 125 feet to 2,500 feet. At flood stage, depths varied from 25 to 60 feet and widths from 2,500 feet to as much as 25,000 feet.

The middle Mississippi, in its length of approximately 195 miles, has an average gradient of 0.6 foot per mile. Discharges at St. Louis, where records exist for over 100 years, show extremes of about 20,000 cubic feet per second in 1933 and 1,300,000 cubic feet per second in 1844, the ratio of these extremes being 1 to 65. Average discharge of the river is approximately 175,000 cubic feet per second. As would be expected, variations in stage are considerable, ranging from minus 6.2 feet in 1940 to a maximum of 41.3 feet in 1844 as measured at St. Louis. Average annual duration of bankfull stage ranges from about 5 days at St. Louis to about 36 days at the mouth of the Ohio River. Practically all of the silt borne in the middle Mississippi originates in the Missouri River; only about 7 percent is contributed by the upper Mississippi. The material of the riverbed is mainly sand and silt, and that of the banks is about 70 percent sand, 20 percent silt, and 10 percent clay.

The river shows a marked tendency to meander in the alluvial valley and is seldom found in the geographical center of the flood plain. Several major local changes in its alinement have occurred since it was first mapped by man. One of the most notable occurred in 1881, a rather cataclysmic one, when the Mississippi broke into the channel of the Kaskaskia River and changed its course so radically that St. Marys, Mo., originally on the main bank, was left 5 miles distant from the river, and Kaskaskia, the first capital of Illinois, was changed from the left bank to the right bank of the river (fig. 1). The entire characteristic of the

FIGURE 1. — Kaskaskia Island before channel change.

river in the vicinity was altered and formed an 11-mile section of relatively young stream in comparison to the river above and below. Great difficulty was encountered in establishing a new and usable trace (fig. 2).

A notable characteristic of the river is the instability of its banks. Experience has shown that there is severe attack on the banks where the radius is less than 6,000 feet and moderate where the radius is not less than 10,000 feet. An increase in velocity and angle of impingement will increase the erosive action. Direction of current in a bend changes as the stage increases. At low water, the point of maximum attack is usually at or near the head of the bend, and as the stage increases, it moves downstream, and at extremely high water it is usually at or near the foot of the bend. As the bank of a river is eroded, a portion or projection is frequently left intact, which, for some reason, is more resistant to attack than the bank adjacent thereto. Here the current is deflected away from the bank, causing a whirl or suction eddy to develop along the bank below the projection. As a result, the attack on the bank is greatly increased in the area immediately below the projection. Lower ends of bank protection also often act as a false point after some caving has taken place below it. Should an eddy current then develop, there is danger of the bank cutting behind the protection work and thus destroying it from below. This type of caving normally accompanies high river stages. The low water caving is normally uniform and fairly free from eddy action.

The rate of progress and the extent of bank caving depend upon the material of which the bank is composed and its drainage, the curvature of the bank and its general direction with reference to the line of attack, the velocity of the current, the position and depth of the channel with relation to the bank, and the stage of the river. The period of greatest caving occurs normally during receding stages. However, the greatest aggregate of caving results from the continuous attack of the river during low and moderate stages. It very rarely happens that both banks cave in the same locality at the same time, and, as a general rule, erosion on

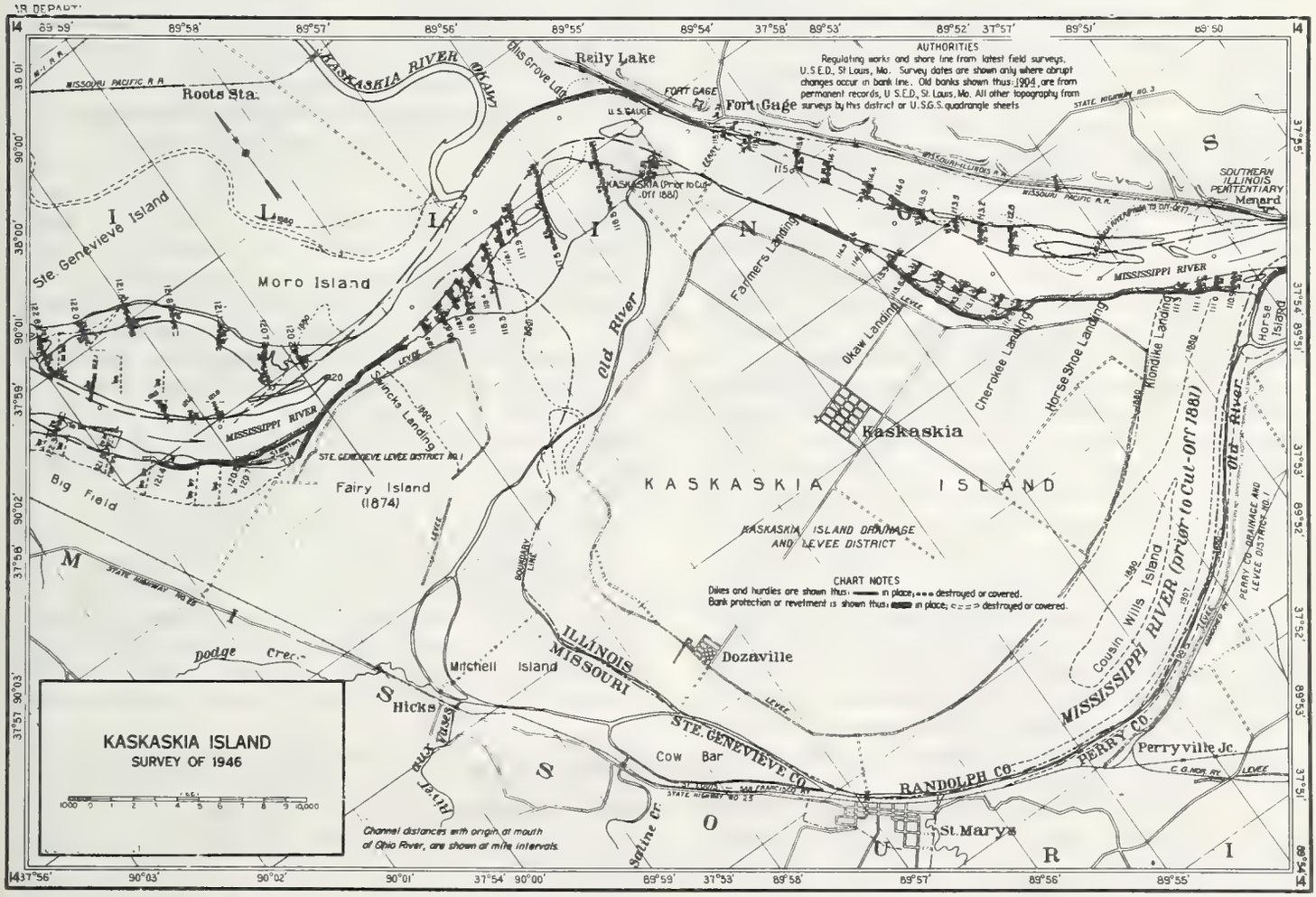

KASKASKIA ISLAND
SURVEY OF 1946
Channel distances with origin at mouth of Ohio River, are shown at mile intervals.
AUTHORITIES
Regulating works and shore line from latest field surveys.
U.S.E.D., St Louis, Mo. Survey dates are shown only where abrupt changes occur in bank line. Old banks shown thus: 1904, are from permanent records, U.S.E.D., St. Louis, Mo. All other topography from surveys by this district or U.S.G.S. quadrangle sheets
CHART NOTES
Dikes and hurdles are shown thus: ——— in place; - - - destroyed or covered.
Bank protection or revetment is shown thus: ▬▬ in place; <= = => destroyed or covered.
KASKASKIA ISLAND DRAINAGE AND LEVEE DISTRICT
Moro Island
Ste. Genevieve Island
Fairy Island (1874)
Big Field
Kaskaskia
KASKASKIA ISLAND
Dozaville
Mitchell Island
Hicks
Cow Bar
Reily Lake
Roots Sta.
Fort Gage
Menard
Horse Island
Perryville Jc.
St. Marys
Cherokee Landing
Horse Shoe Landing
Farmers Landing
Okaw Landing
Shincks Landing
Old River
Cousin Wills Island
MISSISSIPPI RIVER (prior to Cut-Off 1881)
MISSISSIPPI RIVER
ILLINOIS
MISSOURI
STE. GENEVIEVE CO.
PERRY CO.
RANDOLPH CO.
Podge Creek
Saline Creek
KASKASKIA RIVER
Ellis Grove Lake

FIGURE 2. — Kaskaskia Island after channel change.

one bank is followed by an accretion on the other.

The first Federal work for improvement of the Mississippi River was authorized by Congress in May 1824 and provided for the removal of snags between New Orleans and the Missouri River. This project, interrupted during the Mexican and Civil Wars, has been carried on as required ever since. The first work in the nature of permanent improvements, "a pier to give direction to the current of the Mississippi River near the City of St. Louis," was authorized by Congress in the Acts of 4 July 1836 and 3 March 1837.

At that time, the Mississippi River opposite St. Louis was divided into two channels by Bloody Island. For some time, Duncan's Island, immediately downstream and adjacent to the right bank, had been growing rapidly, as well as the shoal connecting it with Bloody Island. During extreme low water, steamboats coming from the South had to use the eastern channel and, turning the head of Bloody Island, drop down to the city landing. The course of the river was tending more and more to the east and threatened to take the channel entirely away from St. Louis. To prevent this, Robert E. Lee, then a Lieutenant in the Corps of Engineers, recommended closing of the eastern channel by a dam from the head of Bloody Island to the Illinois shore and the construction of a longitudinal deflecting dike at the foot of Bloody Island to wash away the shoal connecting it with Duncan's Island. These works consisted of training walls of wood piling, brush, and stone.

The dike from the foot of Bloody Island was constructed in 1838 and proved effective in deepening the channel over the shoal. Robert E. Lee, however, abandoned his first proposal for damming the eastern channel and, instead, recommended construction of a longitudinal dike from a point opposite Venice, Ill. Although useful in preventing the existing unfavorable condition from becoming worse, the dike was not in itself a cure for the condition. Subsequently, a dam was built across the channel east of Bloody Island, made a roadway, and finally closed the eastern channel. The project included revetting the western shore of Bloody Island. (See fig. 3.)

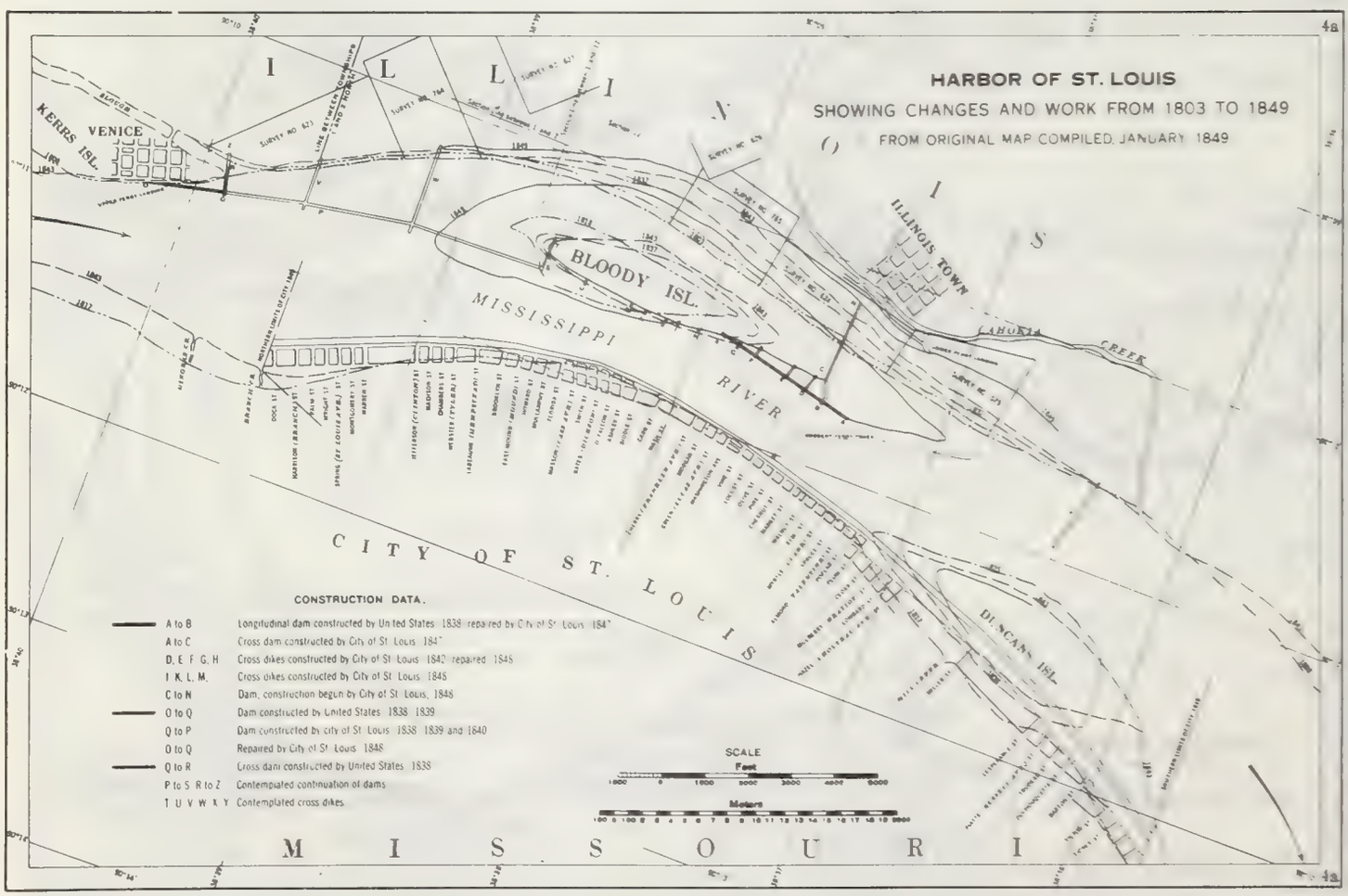

HARBOR OF ST. LOUIS
SHOWING CHANGES AND WORK FROM 1803 TO 1849
FROM ORIGINAL MAP COMPILED JANUARY 1849
ILLINOIS
KERRS ISL.
VENICE
BLOODY ISL.
ILLINOIS TOWN
CAHOKIA CREEK
MISSISSIPPI RIVER
CITY OF ST. LOUIS
DUNCANS ISL.
MISSOURI
CONSTRUCTION DATA.
A to B Longitudinal dam constructed by United States 1838 repaired by City of St. Louis 1847
A to C Cross dam constructed by City of St. Louis 1847
D, E, F, G, H Cross dikes constructed by City of St. Louis 1842 repaired 1848
I, K, L, M, Cross dikes constructed by City of St. Louis 1848
C to N Dam, construction begun by City of St. Louis, 1848
O to Q Dam constructed by United States 1838 1839
Q to P Dam constructed by city of St. Louis 1838 1839 and 1840
O to Q Repaired by City of St. Louis 1848
Q to R Cross dam constructed by United States 1838
P to S R to Z Contemplated continuation of dams
T U V W X Y Contemplated cross dikes
SCALE
Feet
Meters

FIGURE 3. — Channel changes, St. Louis Harbor.

In a report on Harbor of St. Louis, dated 3 February 1844, Captain T. J. Cramm states:

> It is well attested, by observation on the different reaches of a river, whether its banks be of alluvium or of rock, that where the bank is concave, or hollowing toward the river, on that side will be found the deepest water, the strongest current, and the least tendency to deposit; and where the bank is convex, or rounding toward the river, on that side will be found the shoalest water, the weakest current, and the greatest tendency to deposit. . . .,It is greatly to be regretted that the original founders of the town of St. Louis disregarded these circumstances when selecting the site of the city upon a reach where the bank was convex instead of concave toward the river.

There has been and there will probably continue to be difference of opinion regarding the effect of bank revetments. In the past, some opposed them primarily because they did not believe they would be effective and because of the enormous expense they involve, as well as the difficulty of maintaining them. The advocates of revetment, however, argue that by preventing the caving of banks, the supply of materials for building up shoals below is largely reduced and navigation is thereby improved. Congress, in the River and Harbor Act of 1886, imposed on the Mississippi River Commission the following condition: "That no works of bank protection or revetment shall be executed in said reaches or elsewhere until after it shall be found that the completion of the permeable contracting works and uniform width of the high-water channel will not secure the desired stability of the river banks." This limitation was based on the belief that the river, if once regulated, would not scour its natural banks. The Senate Committee in 1898 expressed the opinion that: "In view of the great cost, comparatively great taking into consideration the results, it would be more judicious to limit revetment to levee and harbor protection."

When engineers consider the hydrography and the direction of currents, the Mississippi when low is not the same river as when high, and the problem of permanent improvement involves reconciliation of these differences. The type of structures used in contracting and training an alluvial river ranges from simple expedients using locally available material to carefully planned and constructed permanent works. The comprehensive plan for regulation of the middle Mississippi River was approved 31 March 1881 and provided for continuous improvement downstream from St. Louis, utilizing revetment and permeable dikes, to reduce the width of the river to 2,500 feet. This authority has served as a basis for all subsequent river regulating works.[2] As previously indicated, the earliest plan of improvement in the middle Mississippi consisted of closing dams to cut off sloughs and minor watercourses in order to confine the flow to a single channel. Originally, these closure structures were low, solid dikes consisting of alternate layers of brush and stone to a height of about 8 feet above low water.

Although some success was obtained with solid dikes, experiments with various other types of structures had shown that timber hurdles or screens would produce a more rapid deposition of sedimentary matter. These early experiments indicated that the true role of a dike was to train the river by encouraging it to deposit sediment where desired, thus utilizing the force of the river itself to improve channel conditions. Consequently, when the existing project was adopted, the permeable dike was substituted for the solid structure (fig. 4).

The permeable piling dike consisted of two rows of single piling, well braced and connected by stringers. This type was improved gradually until 1893, when it was replaced by dikes of two or more rows of 3-pile clumps bound together with longitudinal stringers for better distribution of strain, the number of rows of clumps depending upon the depth and swiftness of water. In practically all cases, a foundation mattress of willows or lumber was sunk with stone on the line of the dike to prevent bed scour, the piling being driven through the mattress to a penetration of 20 to 25 feet. (See figs. 5, 6, and 7). This type of dike construction was used in constricting the major portion of the middle Mississippi River into a single navigation channel. Currently, there are 487,500 linear feet of pile dikes operative in the middle Mississippi River. During the early stage of the project, dikes were spaced at distances approximately 2½ times their length. As time went on, the spacing was decreased to two times their length, and at present, spacing is about 1½ times their length. Where pile dikes could not be used, due to underlying rock ledges, crib dikes have been successfully employed (figs. 8 and 9).

Curtain or screen mattresses, hung on the upstream faces of the pile dikes, have been used at times to increase the rate of fill caused by the dikes. These screens have been used extensively since 1940 and have proved very successful, when used in series, in deepening the channel by inducing deposit between dikes (fig. 10). Permeable dikes were built throughout their lengths with tops at about the 20-foot stage, St. Louis gage. Most dikes in the middle Mississippi River acquire a considerable amount of

[2] The existing project for the middle Mississippi River provides for obtaining and maintaining a minimum channel depth of not less than 9 feet, a minimum width of not less than 300 feet at low water, with additional width in bends from the mouth of the Ohio River to the northern boundary of St. Louis; thence 200 feet wide with additional width in bends to the mouth of the Missouri River; to be obtained by regulating works (piling dike and bank protection) and by dredging.

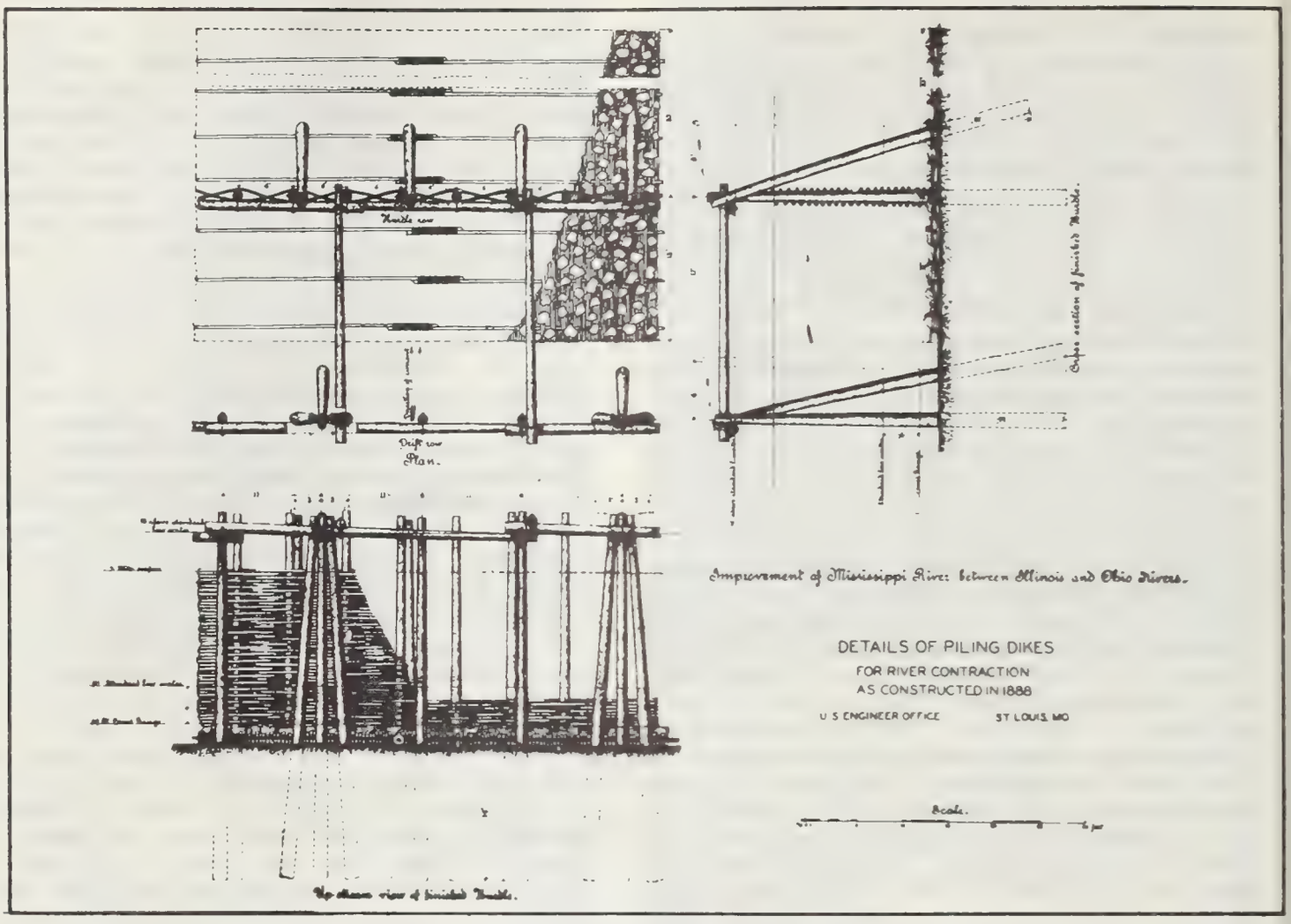

Improvement of Mississippi River between Illinois and Ohio Rivers.
DETAILS OF PILING DIKES
FOR RIVER CONTRACTION
AS CONSTRUCTED IN 1888
U S ENGINEER OFFICE ST LOUIS MO

FIGURE 4. — Early type of permeable dike.

fill behind them before the end of 5 years.

Contracting the river in a short time often creates changes that are too radical for the river to adjust itself to in the time allotted. This often results in the failure of the dikes or entails excessive maintenance work before fills accumulate in the dike system. The success of these improvements requires, in general, construction of a continuous system of regulating works.

Although dikes cause deposition and accompanying accretions, in many instances the dike system may induce a heavy attack on the opposite bank downstream. If unchecked, this erosion of the banks could eventually result in an unsatisfactory alinement and a channel condition as bad as that which the dike system was installed to correct. Furthermore, in an alluvial stream like the middle Mississippi, meandering is inevitable if the velocity is sufficient to erode the banks. This potential condition is everywhere present, so that unless the banks are protected, large-scale caving will occur.

The standard bank protection consists of an underwater part and upper bank part. The underwater part is made of woven mattress of lumber or willows laid parallel to the river bank and sunk to the bed of the river with stone ballast (fig. 11). The upper bank part consists of paving extending from the edge of the bank at about mean low water to approximately the 20-foot stage on the St. Louis gage. It consists of a 10-inch layer of riprap stone from 3 to 150 pounds in weight laid on a 6-inch layer of crushed stone. The bank upon which the stone is to be laid is graded to a uniform slope of about 1 vertical on 3 to 4 horizontal in such a manner that it presents a smooth surface (fig. 12). From 1873 to 1963, approximately 157 miles of bank revetment have been constructed on the middle Mississippi River, of which about 115 miles are currently active. An additional 15 miles of revetment are required to complete the bank stabilization program.

The order of executing work in the improvement of any stretch of river is not fixed and

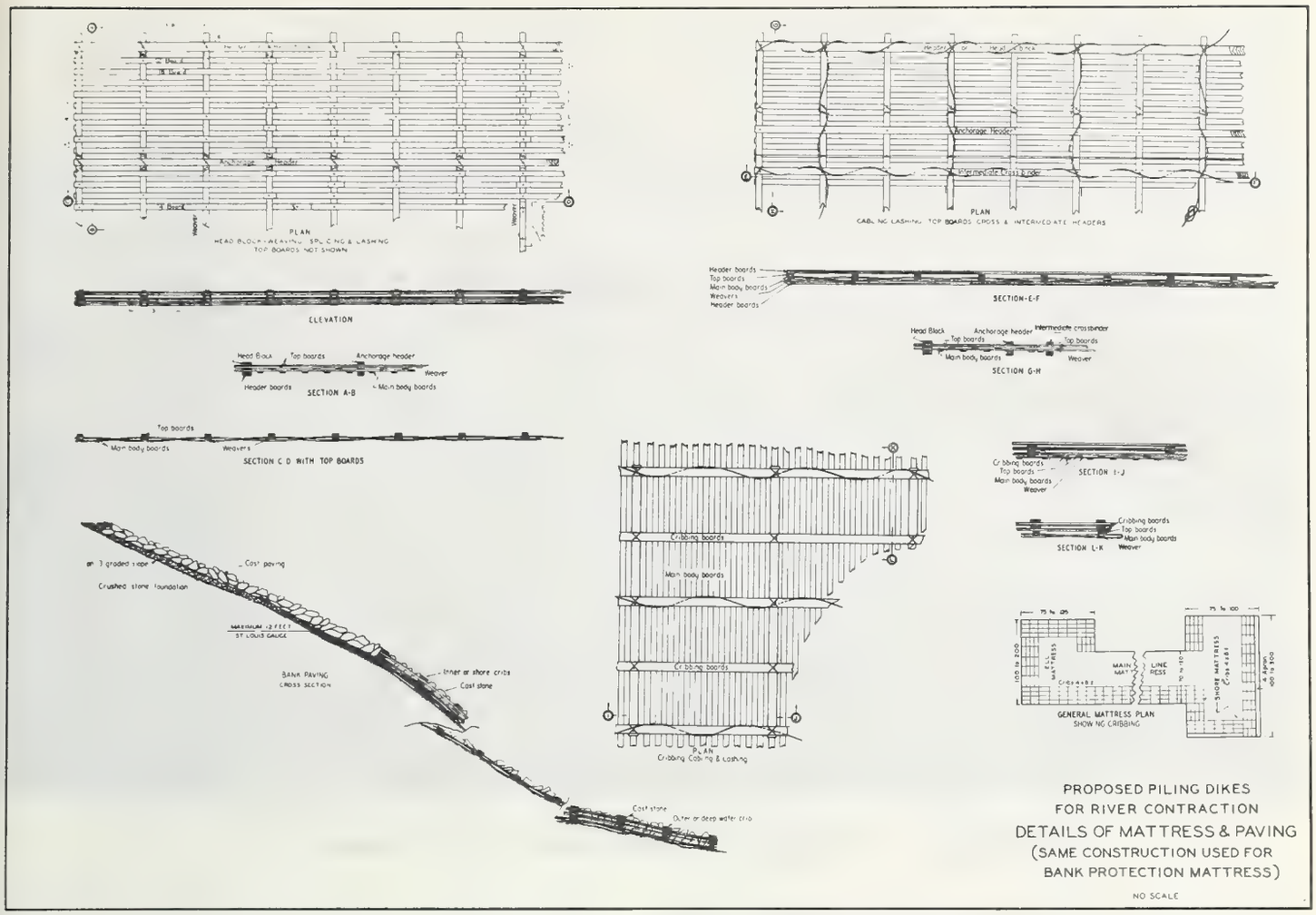


FIGURE 5. — Details of mattress and paving for piling dikes.

depends essentially upon conditions at the site. Work ordinarily should begin with the alinement and fixing of crossings in which the first step would be to fix the outer bank at the lower end of the upstream bend in order to aline the flow of the crossing. This is followed by bank protection to hold the desired curvature at the upstream end of the lower bend. The order of contraction and bank protection in the adjoining bends would depend upon the rate at which the outer bank attains the desired alinement. Normally, work should begin at the upstream end of a reach, construction thus taking advantage of the current. Certainly, bank protection work must proceed downstream because of the method of construction. In the construction of dike systems, however, local conditions may permit construction work to begin at any point or several points simultaneously.

Riverbanks fail as a result of attack from without and within, i.e., erosion or subsidence. Erosion results from direct current action of the flowing water against the banks, from wave wash, and eddy action. The banks of the river are subject to seepage caused by drainage after heavy rains or by falling river stage. Excess moisture seeps through the banks carrying particles with it and causing the material to disintegrate. The training and protective works must be so designed that they will be effective not only at low water but at medium stage. Experience has shown that in the middle Mississippi the ideal shape of a bend, both for bank protection and for dependable navigable depths, is one having a radius of about 14,000 feet. Shaping the bank into a smooth general curve is essential to remove any obstructing points, otherwise eddy action is almost certain to develop and endanger the protection works. In the St. Louis district the tendency of the channel to shift in reaches can best be controlled by construction of contraction works and revetments to introduce a slight sinuosity into reaches, which will tend to fix the deep water channel along the outer banks of the minor bends thus created.

In addition to controlling the trace of the channel at low water, bank protection must be

FIGURE 6. — Mattress and paving for piling dikes under construction.

adequate to prevent a tendency to meander and form new channels at high stages. As a rule, a comparatively smooth flow begins in the crossover immediately downstream from a rock bank and comes into the bend parallel to the bank line to be protected. In this case, the starting point of the revetment is located at the downstream end of the rock bank. Where a crossing approaches a bend at such an angle as to allow direct current attack against the revetment, the revetted bank is difficult to maintain. In such cases, the best results are obtained by contraction works upstream from the bend, to train the channel flow so as to enter the bend on a tangent to the curve. Where the head of the bend does not begin at a rock bluff, the bank line is shaped into the desired curvature by the direct attack of the current which caves away the bank until flow is eased into a parallel position with the bank. In such cases, the starting point for revetment should be selected well above the point where the main current begins to flow along the concave bank. In some instances, a few short spur dikes should be built upstream to take the brunt of the attack in the event the crossing may shift and take position upstream from the head of the revetment.

Major causes of failure of bank protection aside from ice damage are (1) erosion of the bed and saturation of the banks during extremely high river stages — the high velocities tend to pull material through the small openings in lumber or willow mattress thus undermining the revetment to some extent which, together with the over-saturated condition of the bank as the river recedes, causes a sloughing or settlement of the revetment; (2) a sharp, severe attack at various stages that causes bed erosion and undermines the foundation mattress with resultant sliding or sloughing; and (3) extremely low river stages tend to undermine the bank paving because of the increase in velocity at some localities. The composition of the soil has much to do with the stability of bank protection.

Revetment must be sufficiently flexible to permit its conformity to the bank and bed and to allow its adjustment to changes that may develop. The type of revetment and its effectiveness are largely dependent upon the manner in which banks have been prepared to receive it. The principal aim is to obtain a well rounded curvature, leading tangentially to and from straight or curved reaches. Drainage should be provided and the bank graded to a uniform

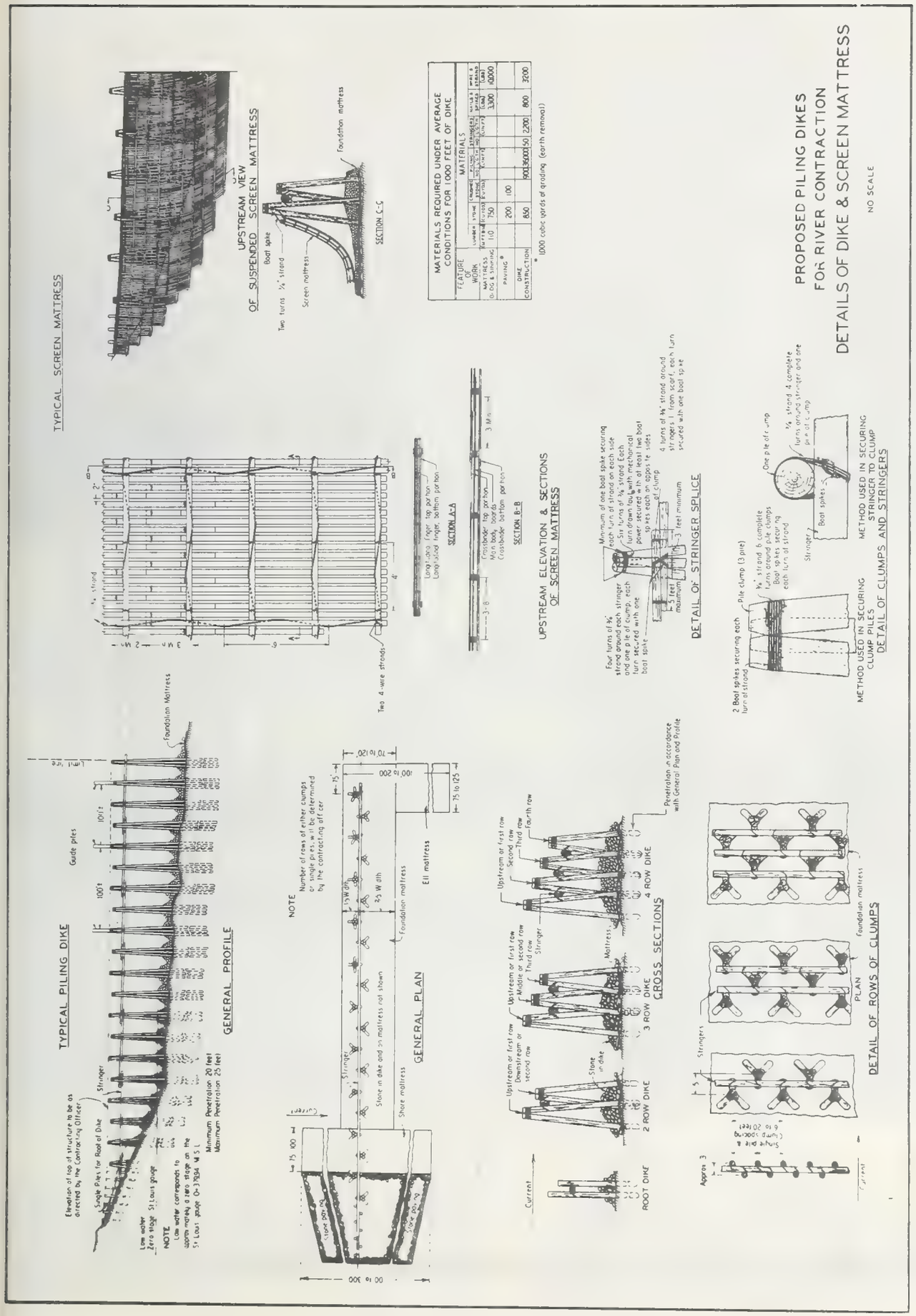

TYPICAL PILING DIKE
GENERAL PROFILE
GENERAL PLAN
NOTE
CROSS SECTIONS
ROOT DIKE
2 ROW DIKE
3 ROW DIKE
4 ROW DIKE
DETAIL OF ROWS OF CLUMPS
UPSTREAM ELEVATION & SECTIONS OF SCREEN MATTRESS
DETAIL OF STRINGER SPLICE
METHOD USED IN SECURING CLUMP PILES
METHOD USED IN SECURING STRINGER TO CLUMP
DETAIL OF CLUMPS AND STRINGERS
TYPICAL SCREEN MATTRESS
UPSTREAM VIEW OF SUSPENDED SCREEN MATTRESS
MATERIALS REQUIRED UNDER AVERAGE CONDITIONS FOR 1000 FEET OF DIKE
PROPOSED PILING DIKES FOR RIVER CONTRACTION
DETAILS OF DIKE & SCREEN MATTRESS
NO SCALE

FIGURE 7.— Details of piling dikes.

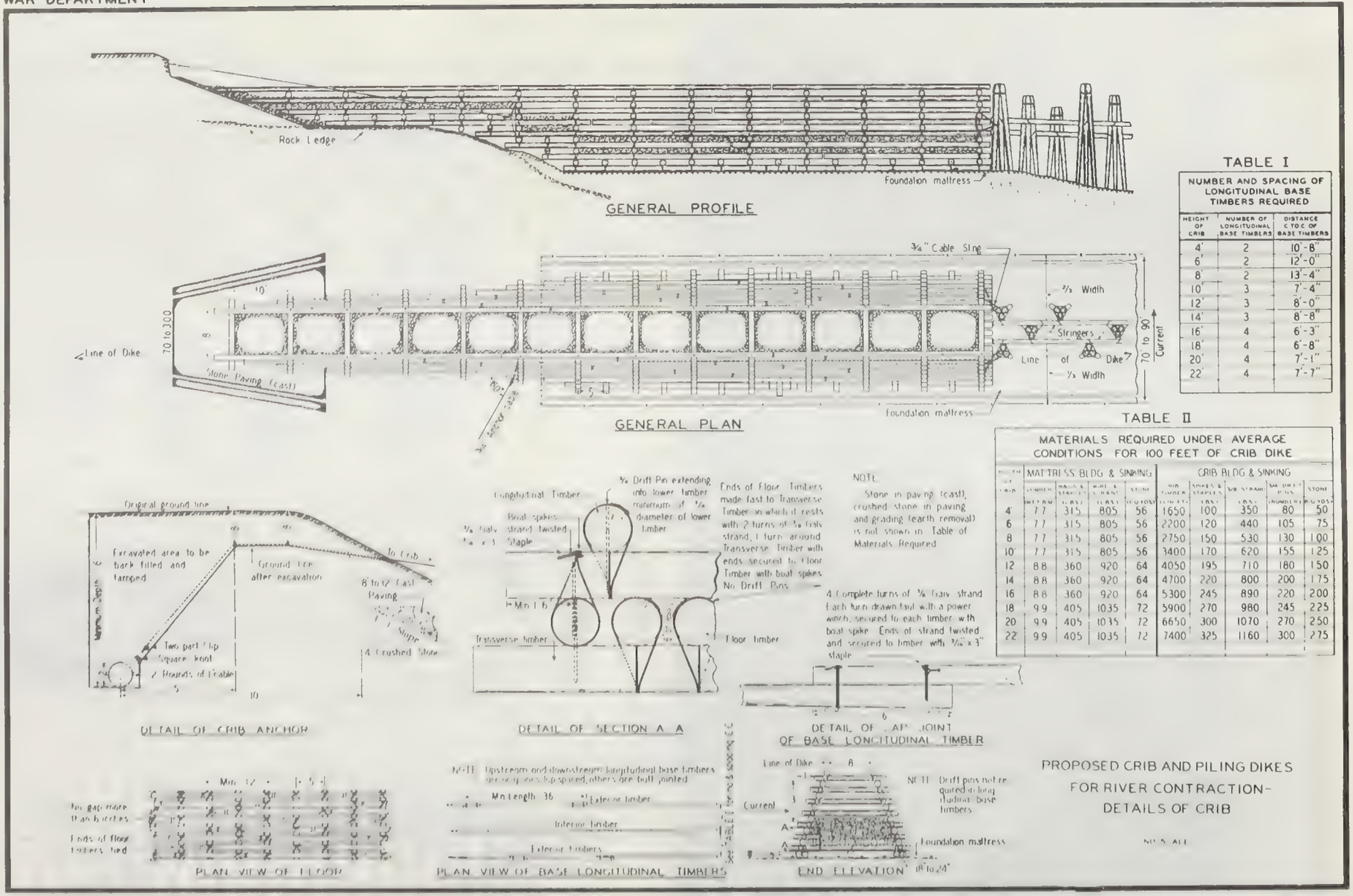

Rock Ledge
Foundation mattress
GENERAL PROFILE
Line of Dike
70 to 300
Stone Paving (cast)
3/4" Cable Sling
1/3 Width
Stringers
Line of Dike
1/3 Width
70 to 90
Current
Foundation mattress
GENERAL PLAN
Original ground line
Excavated area to be back filled and tamped
Ground line after excavation
To Crib
8 to 12 Cast Paving
Slope
4 Crushed Stone
Two part Clip
Square knot
2 Boards of Dunnage
DETAIL OF CRIB ANCHOR
Longitudinal Timber
Boat spikes
1/8 Galv. strand twisted
3/4 x 3 Staple
3/4 Drift Pin extending into lower timber
Min 1 6
Transverse timber
Floor timber
DETAIL OF SECTION A A
Ends of Floor Timbers made fast to Transverse Timber on which it rests with 2 turns of 1/8 Galv strand, 1 turn around Transverse Timber with ends secured to Floor Timber with boat spikes No Drift Pins
NOTE
Stone in paving (cast), crushed stone in paving and grading (earth removal) is not shown in Table of Materials Required
4 Complete turns of 1/8 Galv strand Each turn drawn taut with a power winch, secured to each timber with boat spike Ends of strand twisted and secured to timber with 1/8 x 3" staple
DETAIL OF LAP JOINT OF BASE LONGITUDINAL TIMBER
Min 12
No gap more than 6 inches
Ends of floor timbers tied
PLAN VIEW OF FLOOR
NOTE Upstream and downstream longitudinal base timbers are lap spliced, others are butt jointed
Min Length 36
Exterior timber
Interior timber
Exterior timber
PLAN VIEW OF BASE LONGITUDINAL TIMBERS
Line of Dike
Current
Foundation mattress
NOTE Drift pins not required in long itudinal base timbers
END ELEVATION
PROPOSED CRIB AND PILING DIKES FOR RIVER CONTRACTION- DETAILS OF CRIB

TABLE I

NUMBER AND SPACING OF LONGITUDINAL BASE TIMBERS REQUIRED

HEIGHT OF CRIB	NUMBER OF LONGITUDINAL BASE TIMBERS	DISTANCE C TO C OF BASE TIMBERS
4'	2	10'-8"
6'	2	12'-0"
8'	2	13'-4"
10'	3	7'-4"
12'	3	8'-0"
14'	3	8'-8"
16'	4	6'-3"
18'	4	6'-8"
20'	4	7'-1"
22'	4	7'-7"

TABLE II

MATERIALS REQUIRED UNDER AVERAGE CONDITIONS FOR 100 FEET OF CRIB DIKE

HEIGHT OF CRIB	MATTRESS BLDG & SINKING				CRIB BLDG & SINKING				
	LUMBER BD FT	NAILS & STAPLES LBS	WIRE & CABLE LBS	STONE TONS	LUMBER BD FT	NAILS & STAPLES LBS	3/8 WIRE STRAND LBS	3/4 DRIFT PINS NUMBER	STONE TONS
4'	77	315	805	56	1650	100	350	80	50
6	77	315	805	56	2200	120	440	105	75
8	77	315	805	56	2750	150	530	130	100
10'	77	315	805	56	3400	170	620	155	125
12	88	360	920	64	4050	195	710	180	150
14	88	360	920	64	4700	220	800	200	175
16	88	360	920	64	5300	245	890	220	200
18	99	405	1035	72	5900	270	980	245	225
20	99	405	1035	72	6650	300	1070	270	250
22'	99	405	1035	72	7400	325	1160	300	275

FIGURE 8. — Details of crib dike.

FIGURE 9. — Crib dike under construction.

FIGURE 10. — Screen mattresses for piling dikes.

slope of about 1 on 3. Where the bank material is unstable, the slope may be decreased to about 1 on 5. Because of its flexible nature, riprap is generally more suitable and of longer life than other types of paving.

During the past few years, the number of revetment slides and the high cost of repair showed the need for developing a more stable and economical foundation. Experimental stone foundations were constructed at the base of

FIGURE 11. — Woven mattress for bank protection.

deteriorated revetments. The foundation consisted of approximately 1 ton per foot of ballast or quarry-run stone at the toe of the revetment slope. In every case where stone foundation had been substituted for the lumber mattress, there has been no evidence of deterioration or failure. This type of foundation has proved more stable and less costly than lumber mattress. The method has been adopted and is now being used throughout the middle Mississippi.

Sediment from upstream overbank areas and from bank erosion still are factors in river-regulating works, but their contribution is diminishing. The upper Mississippi has never been considered a heavy silt-laden river, whereas, the Missouri in the past has transported enormous quantities of silt into the middle Mississippi. Observations made since 1910 show that the average silt load at St. Louis has diminished about one-third. This is attributable in a large part to the extensive water resource developments constructed and land treatment measures undertaken in the Missouri valley since the turn of the century. Indications are that we are now entering a stage where impermeable dikes are needed. Acting as baffles, they scour out the riverbed at low stages when very little sediment is in the river. At higher stages, when the silt load is greater, they serve as sediment traps more effectively than the timber dikes. Rock fill dikes are generally cheaper when the height is less than about 23 feet. At greater heights, due to the volume of stone required, pile dikes become more economical. Other factors, however, such as repair and maintenance costs and general effectiveness, must be considered. In some areas, stone dikes have a distinct advantage over timber pile dikes, due to their low cost, greater efficiency, and virtual indestructibility.

FIGURE 12. — Paving for bank protection.

It is evident that the degree, methods, and materials used for river regulation, control, and stabilization were compatible with the specific navigation and flood control needs of each era. The present methods have proved effective and at the lowest possible cost. Materials are readily available and maintenance costs are relatively low. Nevertheless, new and improved methods are constantly being searched for in the light of more intensive consideration of the 12-foot waterway. Improvement over present methods of protection will probably come under the impact of increasing navigation demands and the trend for greater utilization of flood plain areas. Neither the potential of the river nor the needs of man have yet been satisfied.

DOWNSTREAM PATTERN OF RIVERBED SCOUR AND FILL

[Paper No. 46]

By WILLIAM W. EMMETT, *hydrologist*, and LUNA B. LEOPOLD, *chief hydrologist, U.S. Geological Survey*

Introduction

Progress has been made in describing riverbed scour and fill at a given stream section. One needs only the data routinely collected at a stream-gaging station to observe scour and fill at that station. However, similar progress has not been made to determine whether or not the scour and fill observed at a given section extends over a relatively long reach of channel. Gaging stations are generally located too far apart to draw any conclusions as to scour processes between stations. It remains necessary then to establish a sufficient number of cross sections along a channel to describe the downstream pattern of riverbed scour.

This paper presents recent observations of channel scour and fill over relatively long reaches of three streams in the western United States. Separate sections have been devoted to those observations on an ephemeral channel and those on perennial streams.

The authors acknowledge with thanks the

work of Leon A. Wiard, Robert M. Myrick, and other colleagues of the U.S. Geological Survey for data important to the present discussion.

Ephemeral Streams

Geographic Setting and Basic Measurements

The Arroyo de los Frijoles is a sandy ephemeral channel located about 4 miles northwest of Santa Fe, N. Mex. Physiographic and hydraulic characteristics of channels in this locality have been previously studied.[1] Only a summary of those characteristics pertinent to the present paper are presented here.

Normally, flows are the result of runoff from local thunderstorms during the summer months. The flash flood is typical. Peak flow occurs within several minutes after the initial flood wave. Throughout the study reach, the channel increases in size from a rill near the watershed divide to a width of about 100 feet downstream. The bed is composed of medium sand and a moderate amount of gravel. This material extends downward many feet, far deeper than the scour that occurs. The medium sand grain diameter is about 0.5 mm.

Scour and fill data from the Arroyo de los Frijoles were collected by means of scour chains during the 5 years, 1958–62. Determination of the amount of scour and fill using the chain method is a simple procedure. The chains are buried vertically in the streambed with the top link at or slightly above the bed surface. After a flow, the elevation of the streambed is resurveyed and the bed is dug until the chain is exposed. If a scour has occurred, a part of the chain will be lying horizontally. The difference between the previous streambed elevation and the elevation of the horizontal chain is the depth of scour. The difference between the existing bed elevation and the horizontal chain is the depth of fill. If no scour has occurred, the depth of fill is the increase in bed elevation.

Scour chains, each 4 feet in length, were installed along a reach of nearly 6 miles. The location of the chains usually followed the low-water channel. Thus, a flow in any reach of the channel could be expected to pass over the chain location. Over most of the study reach, chains were placed on 1,000-foot intervals. In one reach of 2,000 feet, chains were placed at 100-foot intervals. This spacing was believed sufficient to determine any downstream trend in the scour pattern in this arroyo. At seven of the chain sections, additional chains were installed across the width of the channel and provided an indication of any lateral variation in scour.

[1] LEOPOLD, L. B., and MILLER, J. P. EPHEMERAL STREAMS—HYDRAULIC FACTORS AND THEIR RELATION TO THE DRAINAGE NET. U.S. Geol. Survey Prof. Paper 282–A. 1956.

Observations of Scour and Fill

On the average, there are about three flows down the arroyo each year. Scour and fill data are available for most flows in the 5-year period. However, since some chains were installed before others, an equal length of record does not exist for each chain location. In addition, some chains, usually those in the uppermost or lowermost reaches, were not surveyed after each flow. These missing segments of data disallow a true accumulative value of scour and fill. However, the net change in bed elevation since the time of the initial survey may still be obtained.

By 1959, the majority of the chains had been installed along the arroyo. Scour and fill data for a sample flow, for the year 1962, and for the period 1958-62 are shown on figure 1. For each flow on this figure, the lower dashed line represents the depth of scour and is plotted against distance along the channel. The upper dashed line represents the depth of fill. The heavy solid line represents the net change in bed elevation after scour and fill. The upper part of figure 1 shows the drainage area studied and the general location of the chains by chain number.

The nature of the flash flow does not always flood the entire length of the arroyo with each storm. It may be that the flow-producing rain was so located that only lower reaches received runoff. Or, for a smaller storm near the headwaters, a part, or possibly the entire flow, may be absorbed into the ground by percolation before it reaches a downstream section. A third possibility is that a particular chain section may be left dry or the section has very little scour because it was not in the low-water path of flow. For a single storm then, it is likely that there may be a considerable variation in the recorded depth of scour from section to section. This is further exemplified in the reach centered around station 25,000 feet where the chains are placed on 100-foot intervals. Despite individual variations, a general consistency prevails among the data; that is, at most all sections along the channel, there is a scour and subsequent fill. All flows produce this same pattern with the magnitude of scour primarily dependent upon hydraulic factors of individual flows and these are related to the intensity and total amount of rainfall.

In the data for some flows, short reaches do not indicate a scour. An example is in the reach between stations 2,000 to 3,000 feet for the flow of July 5, 1962 (top profile, fig. 1). In these reaches, the combined action of previous aggradation and of the chain sagging in the vertical had left the top link of the chain some distance below the bed elevation. Thus, scour may and probably did occur, but not to the depth necessary to register on the chain. After

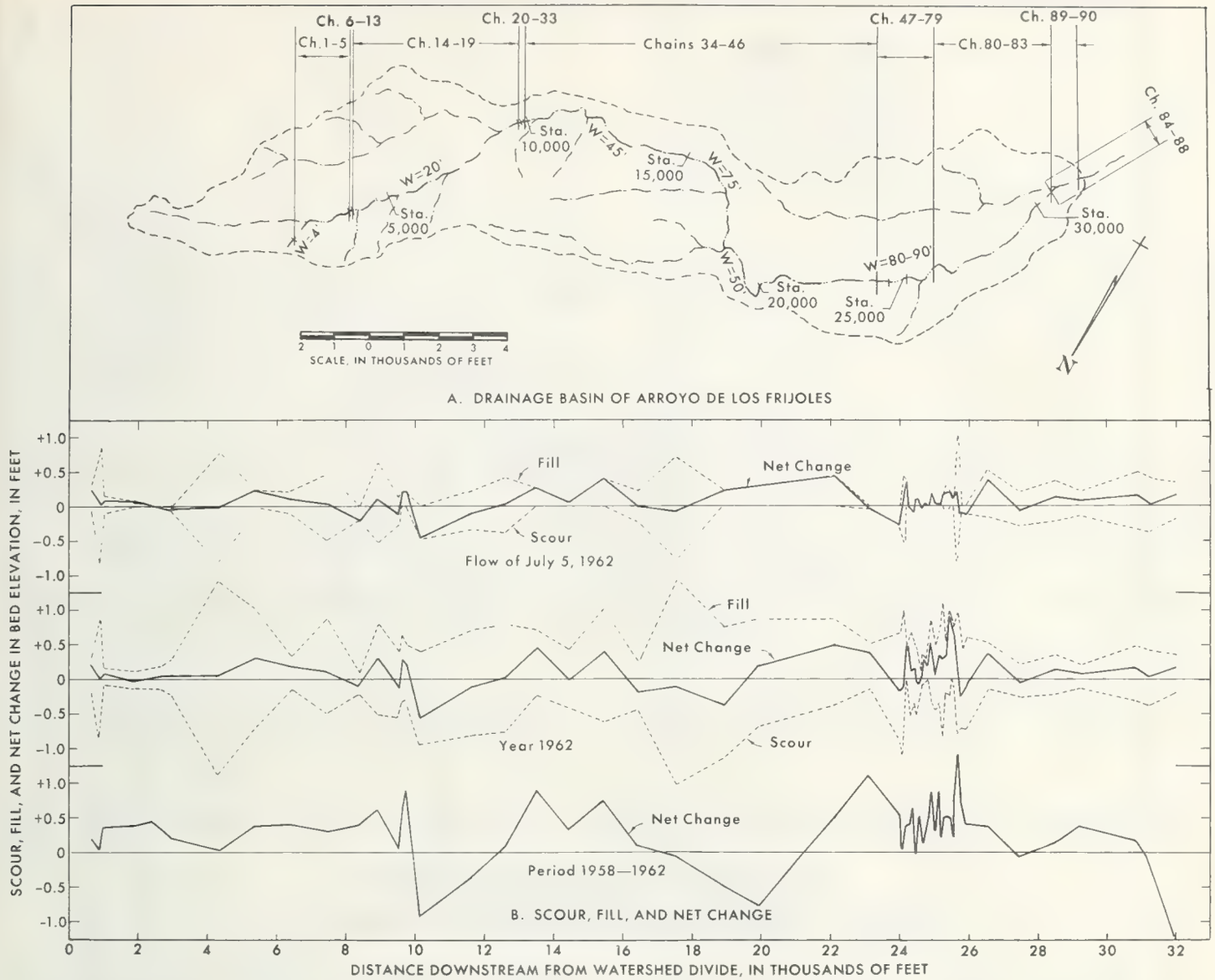


FIGURE 1. — Arroyo de los Frijoles near Santa Fe, N. Mex.

the flow of July 5, 1962, extra length was added to the top of the chains to compensate for the channel aggradation. In addition, closer watch was maintained to check against the chains losing length by sagging.

Three reaches of the channel are the objects of special study and at which flow rates are measured by indirect means, that is, from slope of the high-water marks, estimated roughness, and measured cross-sectional areas of flow. These reaches are centered around stations 2,850 feet, 9,700 feet, and 25,000 feet. It is also within these reaches that the chain sections are located to determine the cross-channel pattern of scour. The mean depth of scour at a section may be determined by averaging the values from the several chains at each of the sections. In a comprehensive report describing various hydraulic and geomorphic processes in the Arroyo Frijoles locale the authors have related these mean values of scour to discharge per unit width of channel. The results of this study are summarized on figure 2. Despite considerable scatter among the data, the mean scour depth appears to be proportional to the square root of discharge per unit width of channel.

An increasing depth of scour is not observed in any single profile as in those shown in figure 1, because during individual flows discharge rarely increases downstream as drainage area increases. This follows from the local nature of most thunderstorm rains and because of the importance of water loss by infiltration into the bed during flows of short duration.

Probably for similar reasons, the depth of scour is apparently independent of the channel

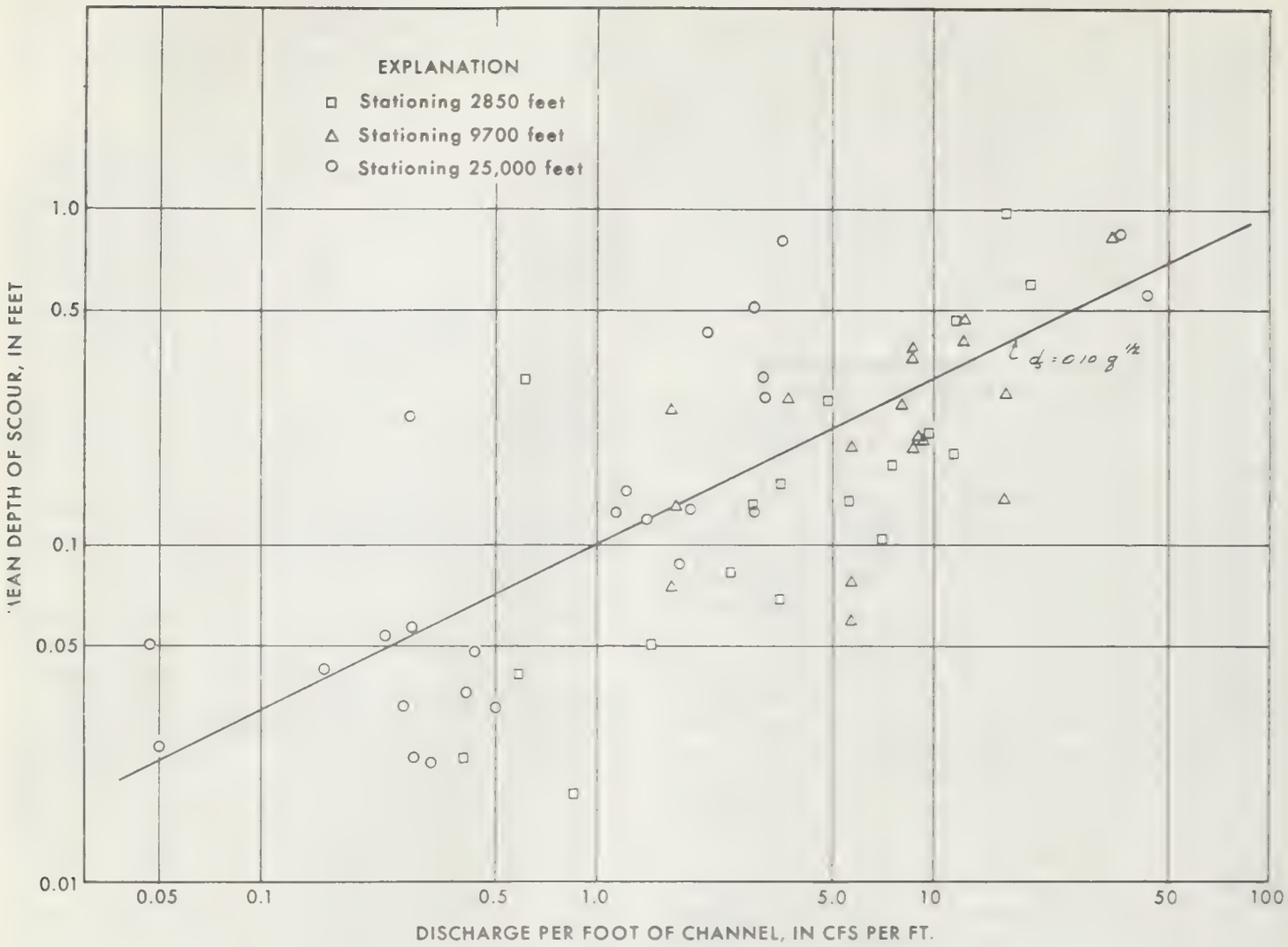


FIGURE 2. — Depth of scour as a function of unit discharge.

width. Channel widths average about 4 feet between stationing 0 to 1,000 feet, 20 feet between stationing 1,000 to 9,000 feet, 45 feet between stationing 9,000 to 13,000 feet, 75 feet between stationing 13,000 to 16,000 feet, 50 feet between stationing 16,000 to 21,000 feet, and 80 to 90 feet from stationing 21,000 feet throughout the rest of the study reach. In addition to the increase of width downstream, local variations occur on the order of threefold to fivefold. No systematic relationship could be established between local depth of scour and the corresponding channel width. Further, since slope decreases downstream as width increases, the magnitude of scour also appears to be independent of slope.

Seven sections provided data to study lateral variation in scour across the width of the channel. Two of these sections, chosen as representative of a particular reach, are illustrated in figures 3 and 4.

Figure 3, illustrating a section at station 25,000 feet, indicates a slight aggradation for the 5-year period. The whole width of channel scours during nearly every flow, but the amount of scour and fill varies across the width.

Figure 4 illustrates a section near station 9,700 feet. Again, considerable variations exist among the yearly records. The 5-year average indicates little net change across the width of the channel. Of particular interest is the lateral distribution in depth of scour. A fourfold decrease in depth of scour is usually noted across the 40-foot width of channel. This difference may be attributed to the higher streambed elevations near the right bank, but even this part of the width scours at times and then fills. The data indicate that some flows may have missed these chain locations entirely and certainly, for all flows, the depths of flow were much shallower on the right side of the channel. The chains at higher elevations are thus expected to indicate less scour.

The scour chains provide an economical and

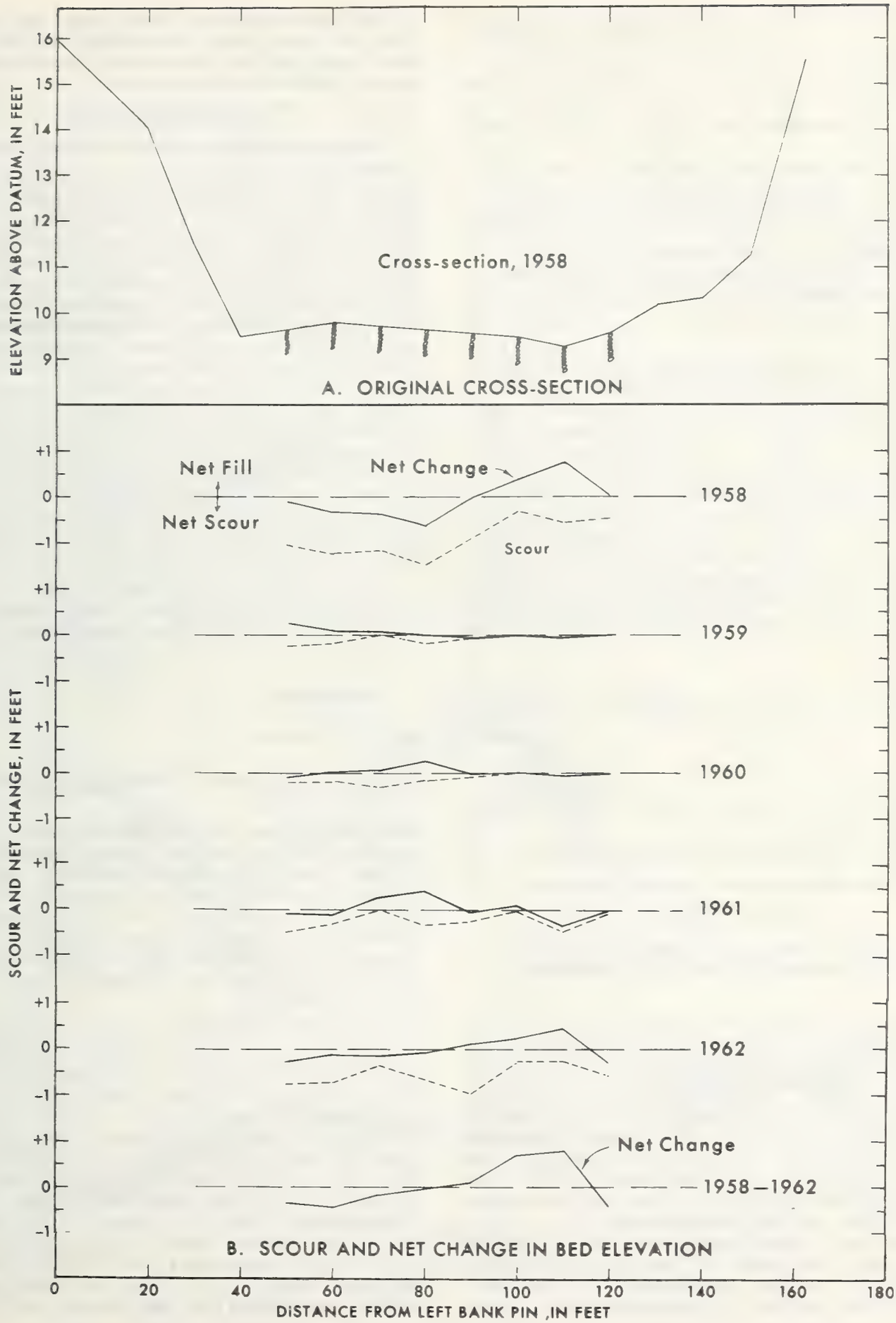


FIGURE 3. — Scour and net change in bed elevation at stationing 25,000 feet for a 5-year period.

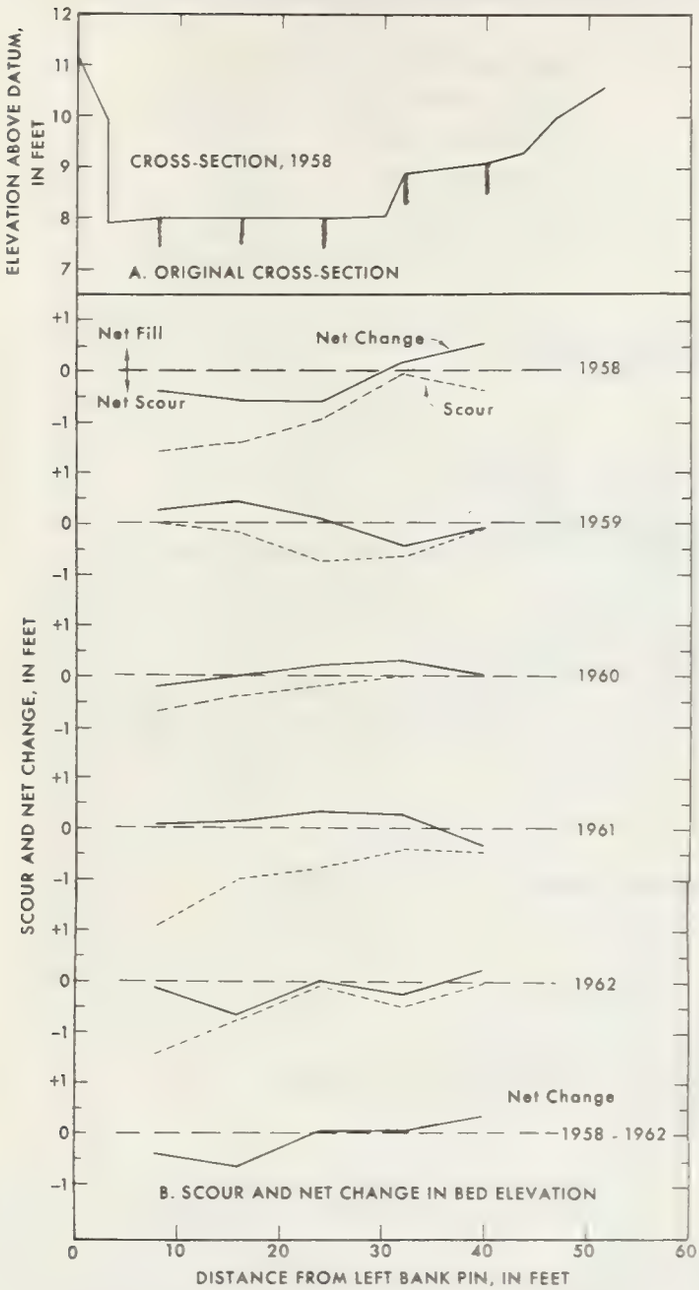


FIGURE 4. — Scour and net change in bed elevation at stationing 9,700 feet for a 5-year period.

simple way of recording net scour and fill for individual flows. It is important that elevations be determined from a reliable bench mark in the immediate vicinity of the chain cross section. We use a 4-foot length of iron reinforcing bar driven nearly flush at each end of the cross section, and one of these is also used as local bench mark for elevation surveys. This iron pin is labeled by a brass tag on which elevation and section number are stamped.

The chain should have a wide or open link so that sand will pack in the chain link. The bottom of the chain is anchored to a rock or a metal head to keep it from pulling out.

The disadvantage of the method is that one does not know when during the flow the scour or fill occurred nor the relative simultaneity of scour or fill at different chains.

Perennial Streams

Geographic Setting and Basic Measurements

The Rio Grande del Ranchos is a small perennial stream on the west slopes of the Sangre de Cristo Range about 7 miles south of Taos, N. Mex. Peak discharges occur in the spring and are normally produced by snowmelt. The study reach consists of a straight reach of 250 feet followed by a curved reach of 700 feet, as can be seen in figure 5. The streambed is predominantly gravel and quite uniform in size from section to section. Median particle size ranges from 21 to 33 mm. with the minimum median size found in the curved reach. The riffle or bar at station 1 + 00 is a predominant feature of the straight reach. A total of seven sequences of pool-riffle occur within the study reach. Channel width at bankfull stage varies from 17 to 36 feet.

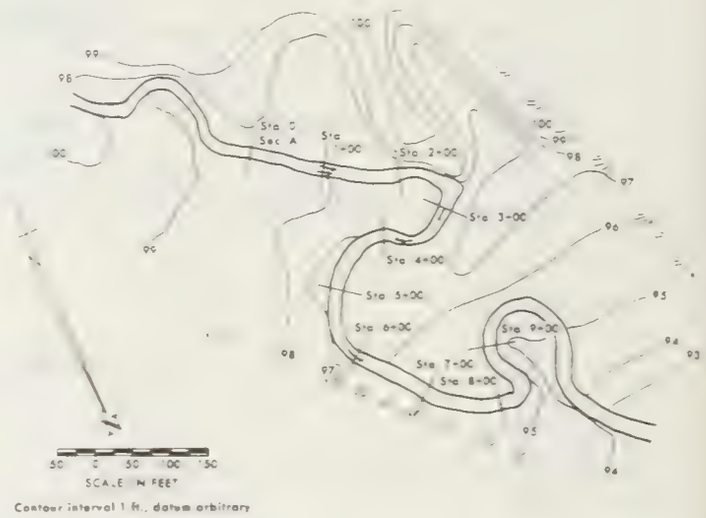


FIGURE 5.—Rio Grande del Ranchos near Taos, N. Mex.

Data were collected by Leon A. Wiard and his associates on the Rio Grande del Ranchos during and immediately after the 1961 snowmelt. A total of 32 measurement stations were established along the study reach. At each measurement station, channel cross sections were obtained by level and rod during the peak flow and at a low flow by depth measurements in connection with velocity observations. Depth measurements were referenced to the earlier survey by known water-surface elevations. High-flow discharge was 130 c.f.s. and low-flow discharge was 25 c.f.s. The data then consist of 32 cross-sections surveyed during spring high

flow (near bankfull stage) and again at low flow in the succeeding summer.

The Popo Agie River is a larger perennial stream in western Wyoming. The study reach is located about 1 mile northeast of Hudson, Wyo., and consists of a curved reach of 2,000 feet followed by a straight reach of 2,100 feet (fig. 6). Peak flow is normally associated with the spring runoff from snowmelt. The streambed is predominantly gravel, ranging in size from fine gravel to large cobbles. Seven sequences of pools-riffles were observed within the study reach. Channel widths at near bankfull stage vary from 86 to 160 feet.

Data were collected at 15 stations along the 4,100-foot reach. Channel cross sections at these stations were obtained during the peak flow in the spring of 1961 and at a low flow later that summer. A second set of data was collected during the rising stage in the spring of 1962 and during a lower, but falling stage that summer. The high-flow observations were obtained by depth measurements referenced to known water-surface elevations. Low-flow cross sections were surveyed with rod and level. High-flow discharge in 1961 was 1,400 c.f.s. and low-flow discharge in 1962 was 310 c.f.s.

Figures 7 and 8 are typical of the cross-sectional measurements obtained. For each stream a section is shown from the curved reach and another from the straight reach.

For both streams and for each measured cross section, the plotted cross sections at high and low flow were superimposed and provided the basis for a scour-fill study. Areas were planimetered from these plots to show the difference in bed between high and low flows.

Low-flow measurements were taken after the high-flow measurements. It is believed, however, that the order in which the measurements were obtained is not of prime importance. Our studies indicate that the cross-sectional configuration of a stream before a high flow is reasonably similar to that some time after the high flow.

The 1961 Popo Agie data and the Rio Grande del Ranchos data represent high-stage measurements obtained during peak flow and the low-stage measurements during the stable low flow. Under these conditions, the changes occurring in streambed elevation were similar in that the bed elevation was scoured during the high flow.

The 1962 Popo Agie data were collected during a different condition of flow. Here, high-stage data were collected during the rising stages of the peak flow and the low-flow data were collected before the stable low flow. In addition, the collection of these data was in the

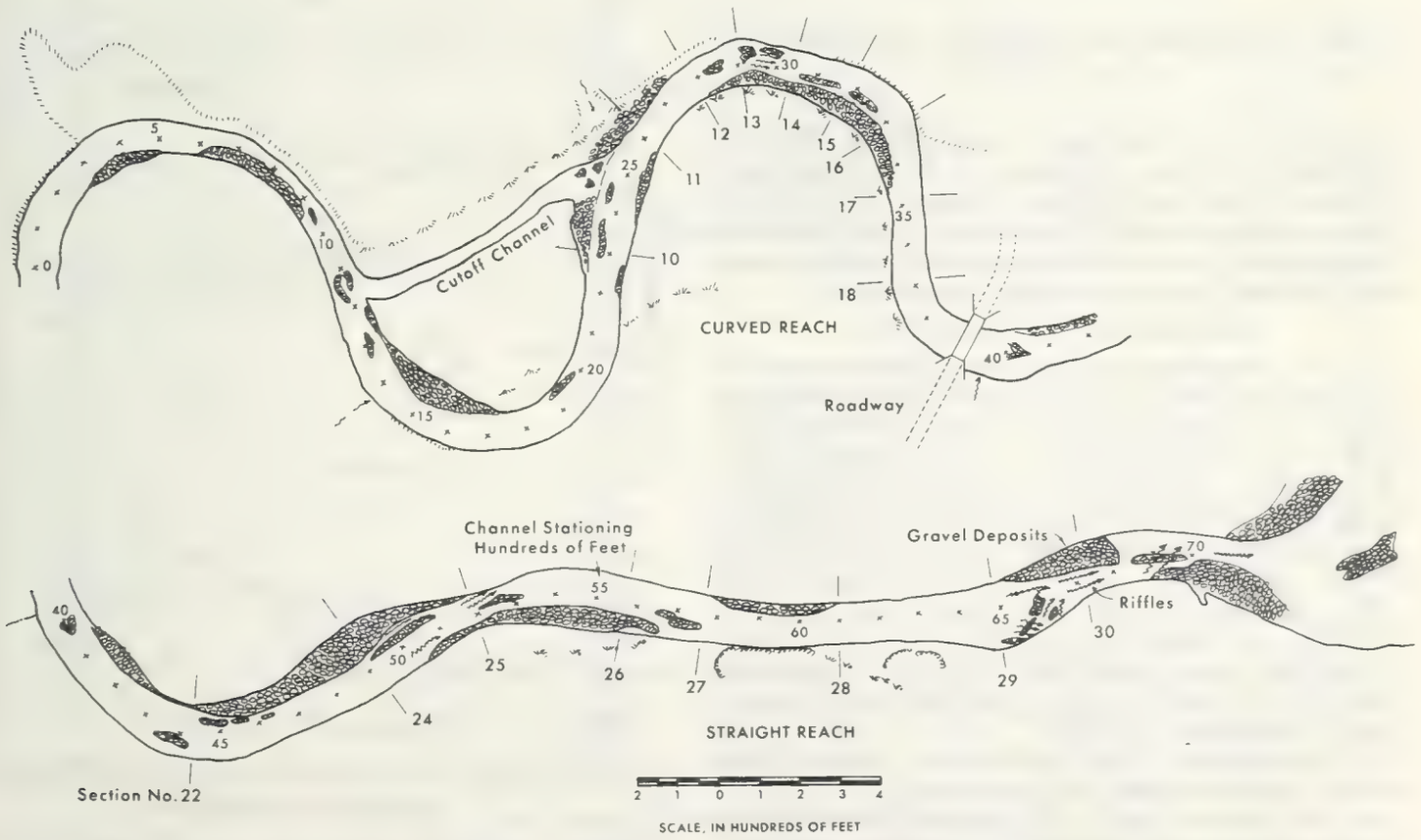


FIGURE 6. — Popo Agie River near Hudson, Wyo.

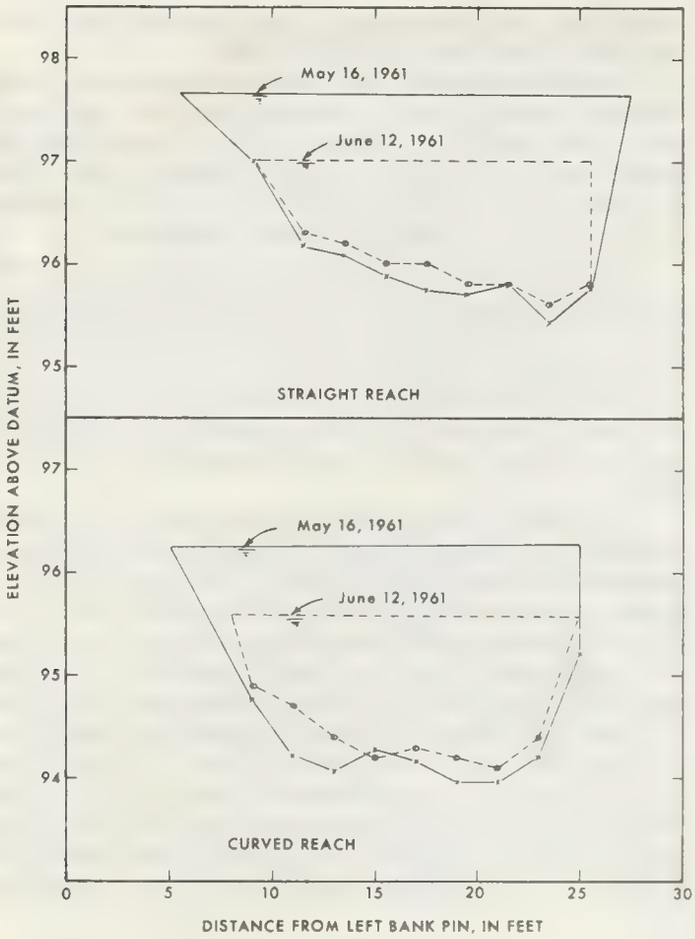


FIGURE 7. — Typical cross sections, Rio Grande del Ranchos, 1961.

aftermath of an unusually large flood from an early spring thaw. Because of streambed conditions left by the flood and because of the river stages at the times of this survey, these data are inconclusive in describing a scour pattern and are omitted from the present discussion.

Observations of Scour and Fill

On figures 9 and 10, profiles of the streambed and water surfaces are plotted against distance along the channel. Below these profiles is a plot of the net changes in cross-sectional areas of the bed and, separately, the cross-sectional area of flow. Values are considered scour if the bed elevations at high flow are lower than those at low flow. Thus, a negative area within this curve represents the total volume of material per foot of length soured from the streambed. A positive area represents the total volume of material brought in from upstream and temporarily deposited as a fill.

It is apparent that at the time of the high-stage measurements a scour generally extended over the entire reach. No marked difference in the magnitude of scour occurs between pool and riffle. Likewise, it appears to make no difference as to whether the measurements were observed in the straight or curved reach. The few sections where fill did occur could not be related to their location within the reach.

It might be expected that scour should be greatest at sections of smallest flow area which, for constant discharge, have the highest mean velocity. To test this hypothesis, flow areas at both high and low stages are plotted at the bottom of figures 9 and 10. No systematic correlation between the flow area and magnitude of scour is apparent. Since discharge is constant at a given stage, this plot may also be considered the mirror image of a comparison of velocity to magnitude of scour.

The absolute magnitude of flow area may not provide an ideal comparison for the influence of area. Some cross-sectional areas at low flow are larger than others at high flow. The relative increase in flow area from a low stage to a high stage would more accurately describe the changes that a particular section experienced. Superimposed as a dashed line on the flow area plot is the relative increase in cross-sectional flow area. Efforts to relate this relative increase to the magnitude of scour were also unsuccessful.

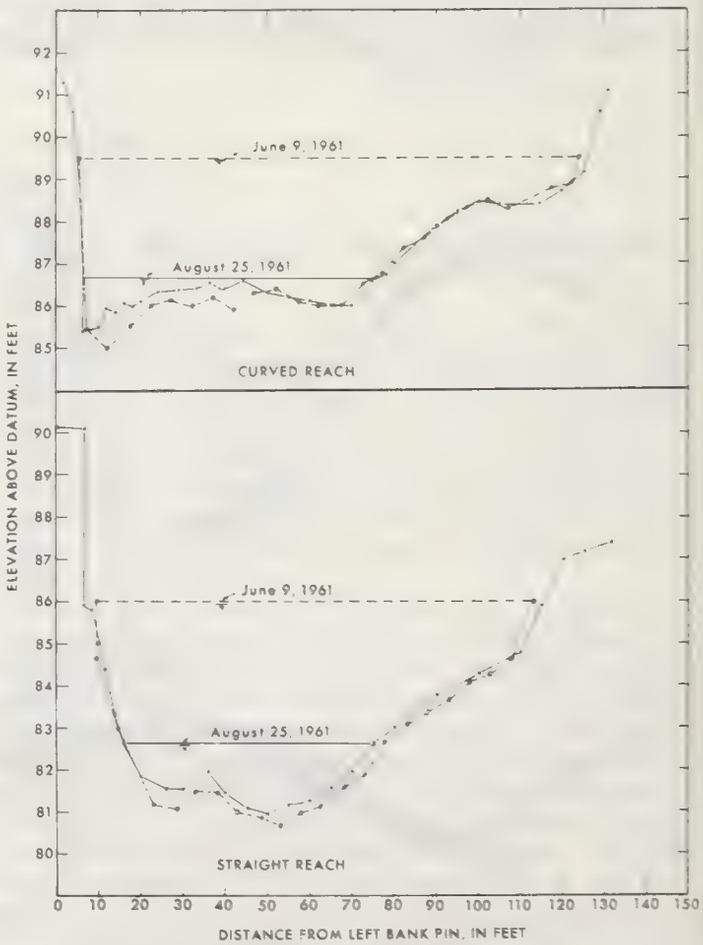


FIGURE 8. — Typical cross-sectional data, Popo Agie River, 1961.

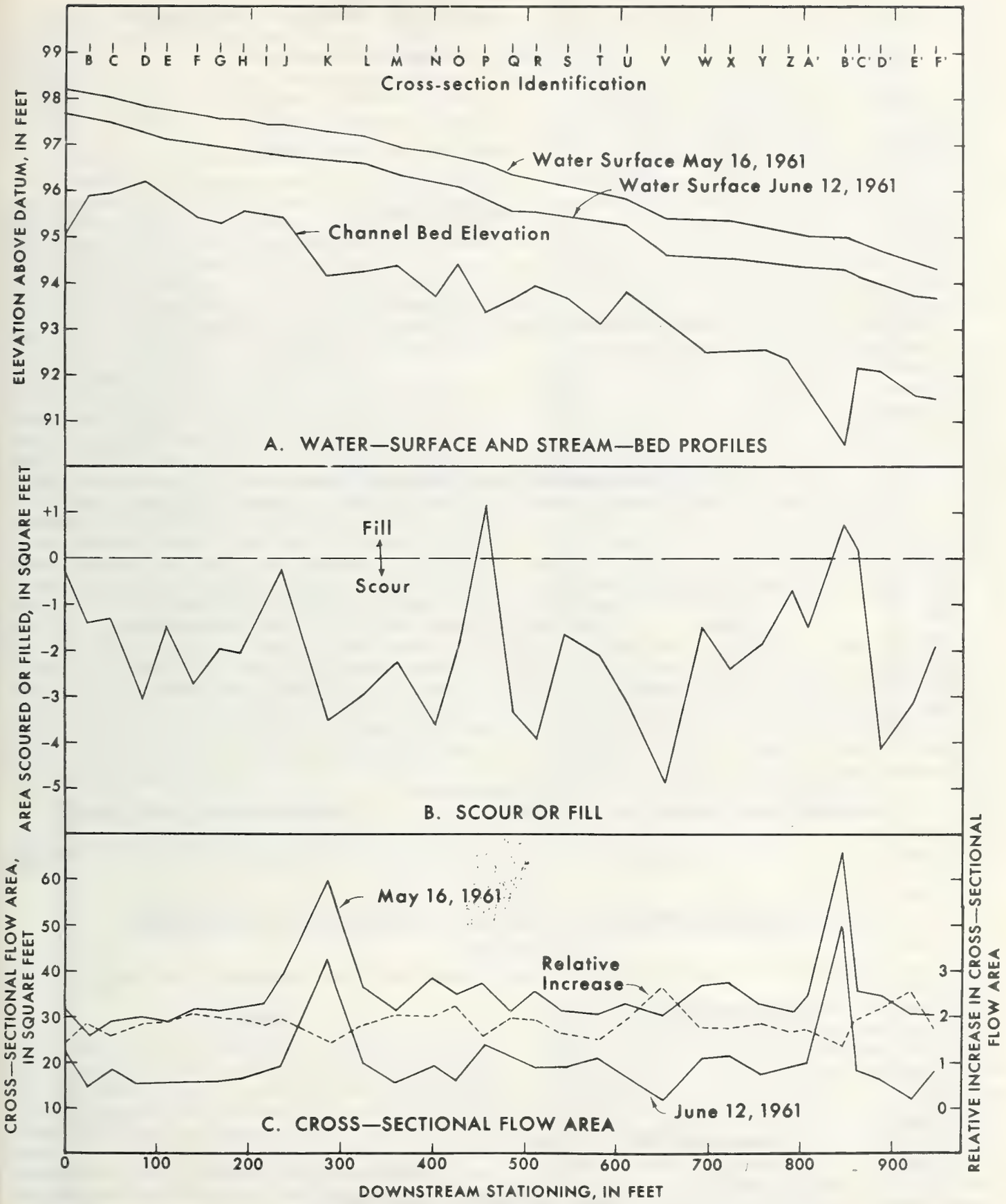


FIGURE 9. — Rio Grande del Ranchos near Taos, N. Mex.

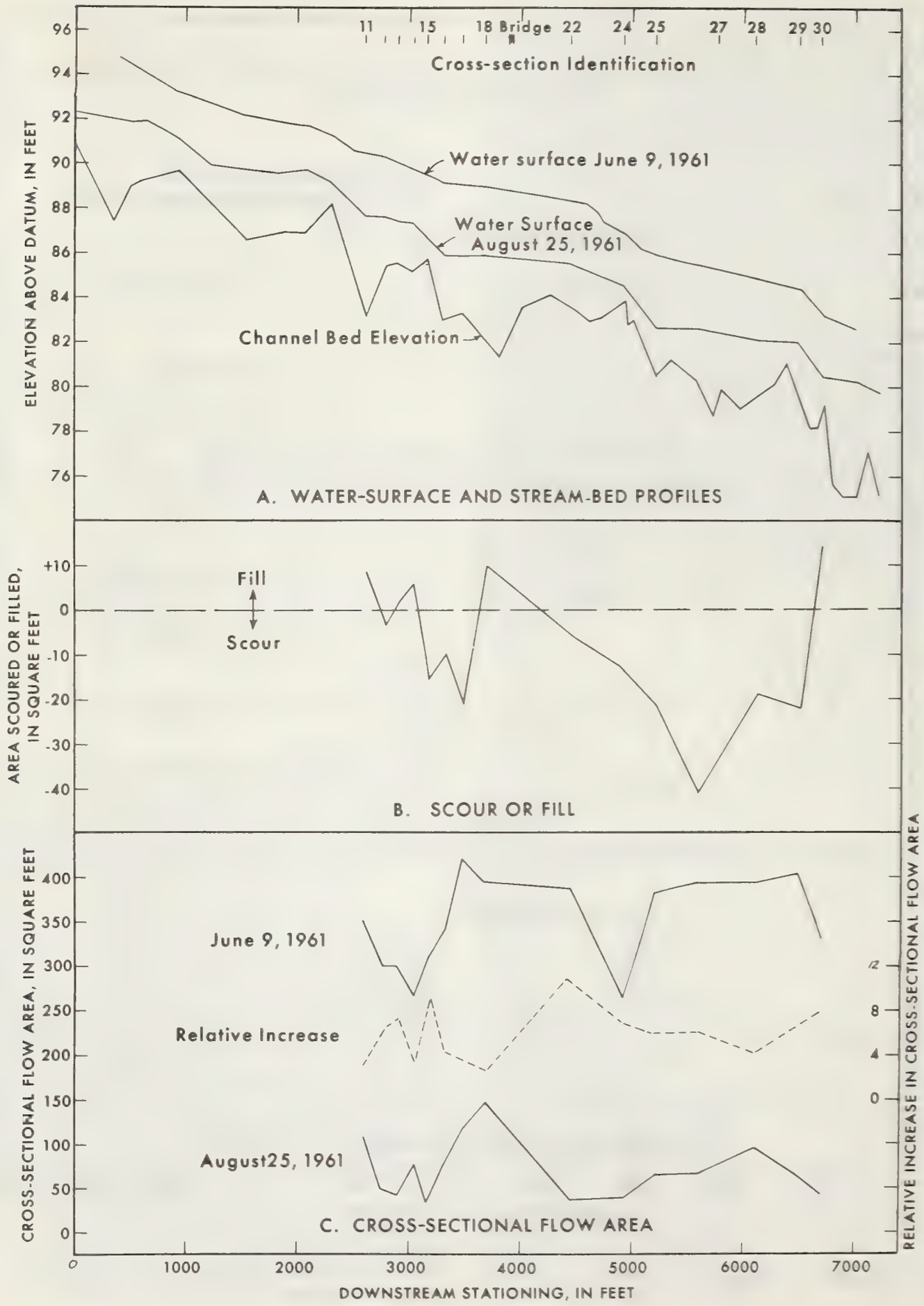


FIGURE 10.—Popo Agie River near Hudson, Wyo.

Similar comparisons were made for other physical dimensions of the stream. Data for channel widths and depths were given the same analysis as for flow areas. No meaningful correlation could be established between these channel properties and the magnitude of scour.

Discussion

At some time during a period of high flow, the channel bottom of fine gravel or sand streams may scour. At present it is not possible to list the characteristics of channels in which this occurs. One tends to think that the scouring action is limited to streams in semiarid areas, but the experience of the U.S. Geological Survey at stream-gaging stations shows that scour occurs during high flow in some rivers in all physiographic and climatic regions.

The extent of the scouring action appears to encompass the entire reach, without regard to any of the physical dimensions of the stream, and applies to pool as well as riffle and to curved as well as straight reach.

Values of scour indicate a large volume of sediment being lifted from the streambed. The data collected might lead one to imagine that this volume of sediment would appear as a block of material being picked up as a unit off the streambed and being carried downstream at a velocity approximating that of the water near the bed. But the interaction or interference between grains tends to reduce the mean speed of the particles which then move at a mean velocity very small relative to the velocity of the flow. This conclusion is not only suggested by the theory of queues but has been verified experimentally by Langbein.[2]

[2] LANGBEIN, W. B. Unpublished data.

[3] LANE, E. W., and BORLAND, W. M. RIVER BED SCOUR DURING FLOODS. Amer. Soc. Civil Engin. Trans. 119: 1954.

There is an apparent disparity between the observed magnitude of scour and rates of accumulation of sediment in reservoirs. Lane and Borland [3] showed that the volume of sediment computed by multiplying observed mean scour depth at a few cross sections by average channel width by channel length is much larger than the observed accumulation of sediment in a downstream reservoir. They argued that the observed scour was nonrepresentative of the whole reach and at unmeasured points fill was simultaneous. Particularly, they said, scour occurs only at the narrow sections of rivers where observations are usually made, whereas at wide sections, if observations were available they would show simultaneous filling.

The present data dispute that argument. We measured, albeit in reaches of modest length, scour in narrow as well as wide sections, in riffles as well as pools, and in curves as well as straight reaches. The observations are deficient, as pointed out earlier, in not showing what was happening in the reach at any one instant of time. Nevertheless, if scour followed by fill in a narrow section were contemporaneous with fill followed by scour in wide sections, the latter should show no scour or much less than that occurring elsewhere. Rather, the magnitude of scour was of the same order in all types of sections.

We interpret the process, then, quite differently from Lane and Borland. Scour is associated with dilation of the grain bed through the scour depth, but individual particles may move intermittently, and at a speed much less than that of the water. The volume of material scoured and moved may be large, but because of its low mean speed downstream, the whole volume does not move entirely out of a long reach but, in effect, is shifted downstream only a limited distance.

REPUBLICAN RIVER CHANNEL DETERIORATION

[Paper No. 47]

By WILLIAM L. NORTHROP, *chief, Water Control Section, U.S. Army Engineer District, Kansas City*

Synopsis

A loss of two-thirds of the operational channel capacity of the Republican River below the Trenton and Harlan County Dams since design of the projects has caused increased damage from local flooding and, if not rectified, will reduce the effectiveness of flood control storage provided in the reservoir system. A study was inaugurated to determine the cause of the loss and, if practicable, to recommend corrective measures. Many factors were involved, such as the sediment load, bed material, uncertainties in streamflow records due to shifting channels, a long drought period, reservoir regulation effects, agricultural encroachments, and an unexpected clogging of the channel by woody growth. A summary of the sediment and hydrologic factors is presented. It is concluded that, when assuming the initial operational capacity, the implications of a temporary increase in channel capacity due to a cycle of wet years were overlooked. Channel losses were caused principally by agricultural encroachment and clogging by phreatophytes. It appears practicable to restore that part of the channel capacity lost due to clogging by mechanical

removal of the woody growth and prevention of regrowth. Retention of woody growth on selected banks, construction of some cutoffs, and installation of light bank protection, will be required to control caving banks.

Introduction

Loss of operational channel capacity on the Republican River has caused increased damage from floods originating below the Harlan County and Trenton Dams and, if not rectified, will reduce the effectiveness of flood control storage provided in the reservoirs. Operational capacities of 10,000 c.f.s. below Trenton Dam and of 15,000 c.f.s. below Harlan County Dam were assumed in reservoir design studies on the basis of discharge data and flood damage information available in the 1940's. Subsequently, in making experimental releases from both projects in 1957, it was found that flows exceeding 3,500 and 4,000 c.f.s., respectively, would cause initial damage. Investigations have now shown that: (1) the original determinations of capacity were optimistic, in that channel enlargements due to a wet cycle, 1935-47, should have been discounted; (2) woody growth began to establish itself in the sandy bed of the stream in the early 1950's, simultaneously with the beginning of a 4-year drought period and, contrary to past history, had grown to sufficient height by 1957 to partially block the channel; and (3) farm crops planted to lower levels since 1950 have reduced the stage of initial flood damage by more than 1 foot. The total length of caving banks between Harlan County Dam and the mouth of the Republican River has been reduced from 121 miles in 1951 to 20 miles in 1961 due primarily to woody growth established on the banks during the drought period; however, the clogging of the channel by woody growth and increasingly unfavorable channel alinements seem to have aggravated some of the remaining caving bank areas.

Basin Description

Geography

The Republican River rises in northeastern Colorado and flows northeast, east, and southeast through Colorado, Nebraska, and Kansas to join the Smoky Hill River at Junction City, Kans., to form the Kansas River. Principal features are shown in figure 1. The basin is

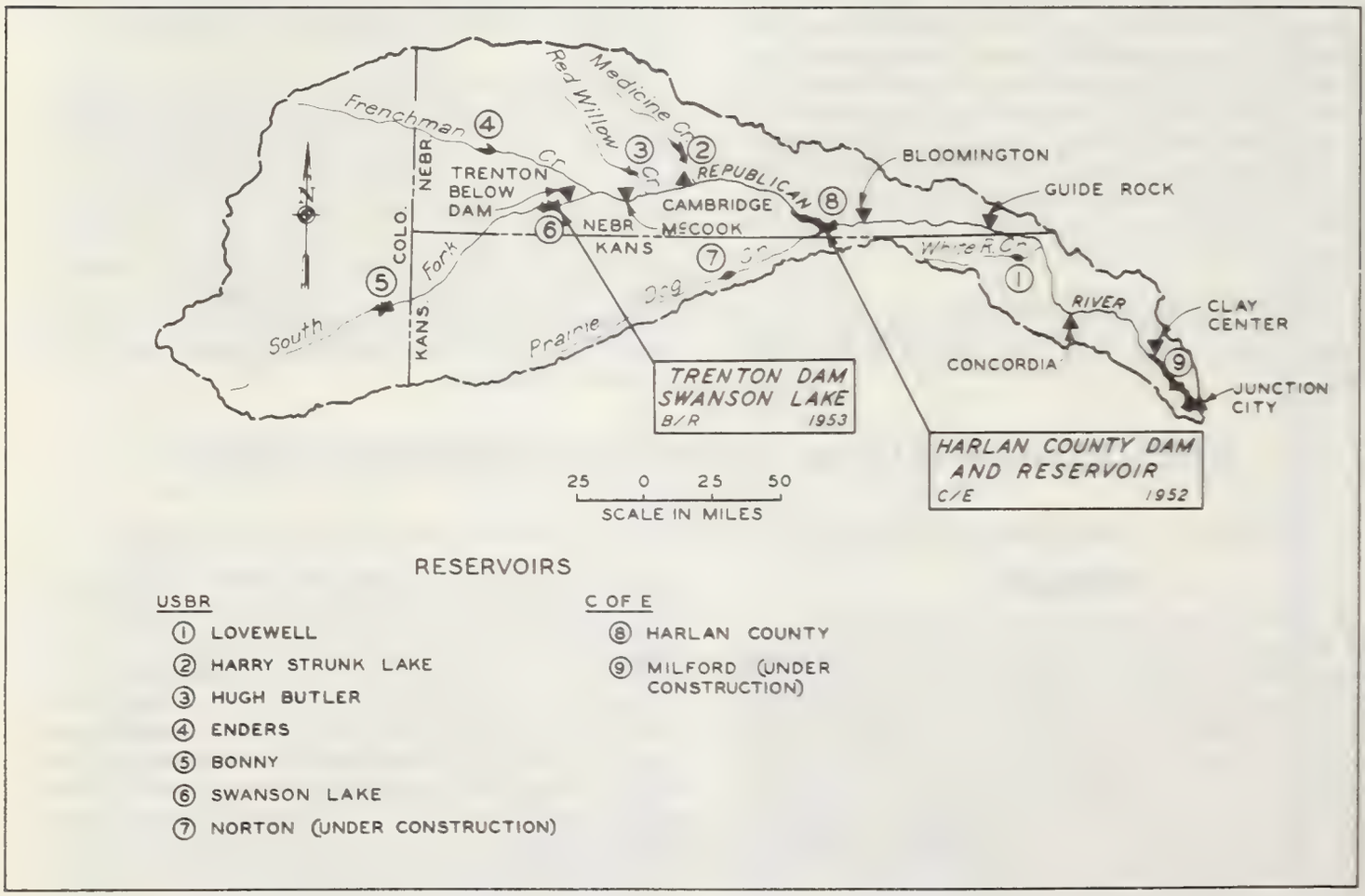


FIGURE 1. — Features of Republican River basin.

nearly 400 miles long from west to east, and it has a total area of 24,955 square miles. The western part is fan-shaped up to 120 miles wide, and it makes up about four-fifths of the basin area. The eastern part is a relatively narrow corridor, 160 miles long and 20 to 30 miles wide. The gross drainage area includes sandhills of the upper basin, estimated to cover 7,170 square miles, which do not contribute surface runoff. The main stem valley below Trenton Dam is relatively wide and a large percentage is used for agricultural purposes. An elaborate irrigation system associated with the reservoir projects is in the final stage of development. The Republican River is a heavy sediment bearing stream with a shifting sand bed and banks of easily erodible soil. Woody growths, principally of willows and cottonwoods beginning about half bank level and extending outward and onto back channel areas, have been recognized as characteristic growth. Caving banks on the outside of bends have plagued valley residents since the land was brought under cultivation.

Climate

Almost all of the Republican River basin lies west of the 97th meridian, which is generally accepted as the boundary between the semiarid plains to the west and the more humid region to the east. The climate is characterized by extremes. The mean temperature is 51° F., but extremes of minus 43° and plus 118° F. have been recorded within the basin. The normal growing season ranges from 150 days in the western section to 190 days in the eastern. The annual precipitation is 16 inches in Colorado and increases easterly to 30 inches in the lower basin. The rainfall rarely is well distributed, as there are localized cloudbursts and prolonged droughts in the weather picture. In May 1935, a maximum rainfall of 24 inches in less than 24 hours was reported at one location in the upper Republican River basin. At other times, droughts and duststorms have prevailed. The average annual snowfall of 27 inches normally does not contribute to flood runoff.

Streamflow

The variable climate produces wide fluctuations in streamflows. Observed discharges of the Republican River at Clay Center, Kans., during the 1934 drought were 1 c.f.s. on 9 to 11 August; and, within a year, the discharge reached 195,000 c.f.s. on 3 June during the 1935 flood. A recurrence of such extremes is not expected now that there is an appreciable degree of upstream reservoir control, but these values illustrate the climatic conditions that prevail in the basin. Presently, there are 48 stream-gaging stations operated in the basin by the U. S. Geological Survey. The average streamflow at the Clay Center gage (98 percent of the gross drainage area) is 1,079 c.f.s., or the equivalent of 0.57 inches of annual runoff.

Basin Development

A comprehensive plan for flood control and irrigation is nearing completion in the Republican River basin. The U.S. Bureau of Reclamation has completed five multiple-purpose reservoirs in the upper basin areas, one on White Rock Creek in the lower basin, and has a seventh project under construction. The U.S. Army Corps of Engineers has completed the Harlan County Reservoir and has the Milford Reservoir under construction. Extensive irrigation projects in the valley of the main stem and tributaries are associated with all reservoir projects. The two principal reservoirs directly related to the channel capacity problem are described below.

Harlan County Dam

Harlan County Dam and Reservoir on the Republican River in south-central Nebraska has a flood control allocation of 500,000 acre-feet and an irrigation, conservation, and sediment reserve pool of 350,000 acre-feet. The dam controls the runoff from an uncontrolled area of 9,277 square miles below the Trenton, Enders, Red Willow, and Medicine Creek Dams of the Bureau of Reclamation.

Trenton Dam

Trenton Dam on the Republican River was completed in 1953. The conservation pool and sediment reserve of 120,200 acre-feet form Swanson Lake, which has an area of 4,974 acres. In addition, the reservoir has a flood control allocation of 133,800 acre-feet. The gross drainage area at the damsite is 7,766 square miles, of which 5,891 square miles lie below Bonny Reservoir.

Flood Damages

Since the turn of the century, the Republican River basin has experienced many damaging floods. By far the most devastating flood of record occurred in 1935, causing an estimated $9,000,000 in damage at that time and level of development. This flood originated in the headwater areas and converted a relatively narrow channel, meandering through a pleasant green valley, to a sandy waste 1,000 to 3,000 feet in width. The channel damage decreased as the flood crest attenuated downstream. In 1947, a flood, originating principally between the Trenton damsite and the Harlan County damsite, also caused extensive damage. Since construction of the reservoir projects, the principal flood damage potential has been eliminated. However, floods originating below the dams have caused average annual damages of about $300,000 along the main stem since 1953.

Reduction in Channel Capacity

At the time Harlan County and Trenton Dams were being designed and constructed, the Republican River had a relatively large channel capacity and gave some indications of further increase. A review of the discharge measurements at pertinent stations indicated a capacity below Harlan County Dam of 15,000 to 20,000 c.f.s. The nominal flood stage at Bloomington, which was then 1.2 feet higher than at present, was apparently used to establish the upper limit of regulated reservoir releases. Likewise, a value of 10,000 c.f.s. was used below Trenton Dam. After the reservoirs were constructed in the early 1950's, a severe drought period followed and it was not until the spring of 1957 that the occasion arose for making substantial reservoir releases. At this time, less than a third of the assumed channel capacity was found to be available below Harlan County Dam and limitations were equally serious below Trenton Dam.

Discharge Relations

Stage-discharge relations developed from U.S. Geological Survey streamflow measurements at Bloomington, Nebr., shown on figure 2,

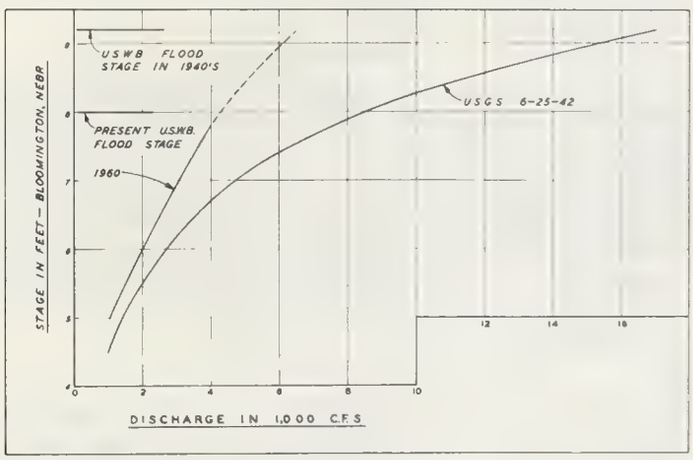


FIGURE 2. — Stage-discharge relations in 1942 and 1960, Republican River near Bloomington.

illustrate generally the changes that have been experienced in channel capacity between Trenton Dam and the Kansas-Nebraska State line. Deterioration below the State line gradually diminishes. It is recognized that single-line rating curves at all Republican River stations are an oversimplification, due to shifting channels and occasional forced movement of gaging equipment. Nonetheless, by analysis of flow records and field observations, it has been established that the usable channel capacity for nondamaging reservoir releases below Harlan County Dam, which had been assumed to be 15,000 c.f.s. in 1943 to 1945, was reduced to 4,000 c.f.s. in 1957. It is evident that about half the 11,000-c.f.s. loss in operational capacity is due to lowering the flood stage a little more than 1 foot since that time, and the remaining loss is an actual reduction in the carrying capacity of the channel. The reduced flood stage reflects the extension of agricultural crops to lower land during the dry years of the 1950's. This was accelerated, no doubt, by the construction of Trenton and Harlan County Dams, which many people in the valley assumed would eliminate all flood problems and permit cultivation of previously unused low areas.

Flow Velocities

Reductions in channel capacity, as indicated by an analysis of the rating curves over a period of time, can be observed also by a close inspection of measured velocities at the discharge stations. At two stations, Bloomington and Guide Rock, measurements at the same sections were made before and after the deterioration in channel capacity. The situation at Bloomington is shown on figure 3. Maximum velocities have been reduced from 8.7 to 6.7 feet per second and the area of high velocities has been reduced substantially. In areas now blocked by woody growth, velocities have been reduced from 5.0 to 1.0 feet per second. Discharges at approximately the same gage height were 8,130 c.f.s. in 1951 and 3,760 c.f.s. in 1961.

Channel Areas

Surveys of degradation ranges below the reservoirs indicate considerable change in channel configuration. Some sections have increased in total area and others have decreased. In most sections, a discernible trend is toward a well-defined narrow central channel and an increase in low island and shore bar heights. Summary data on the ranges below Harlan County Dam indicate very little average change in cross-sectional area for the entire reach between 1951 and 1957, although individual sections have changes up to 15 percent of the area (table 1). Changes were fairly well distributed between the upper and the lower half of the sections. Similar data for the ranges below Trenton Dam, shown on table 2, indicate a net loss between 1949 and 1958 of 7 percent in the average total channel area and a loss of 19 percent in the average lower half. Of the entire average loss 87 percent occurred in the bottom half of the channel and loss at one location was 40 percent of the entire section.

Woody Growth

In 1952, a profuse growth of willow seedlings, mixed with cottonwood seedlings in some places, was observed on many islands and shore bars in

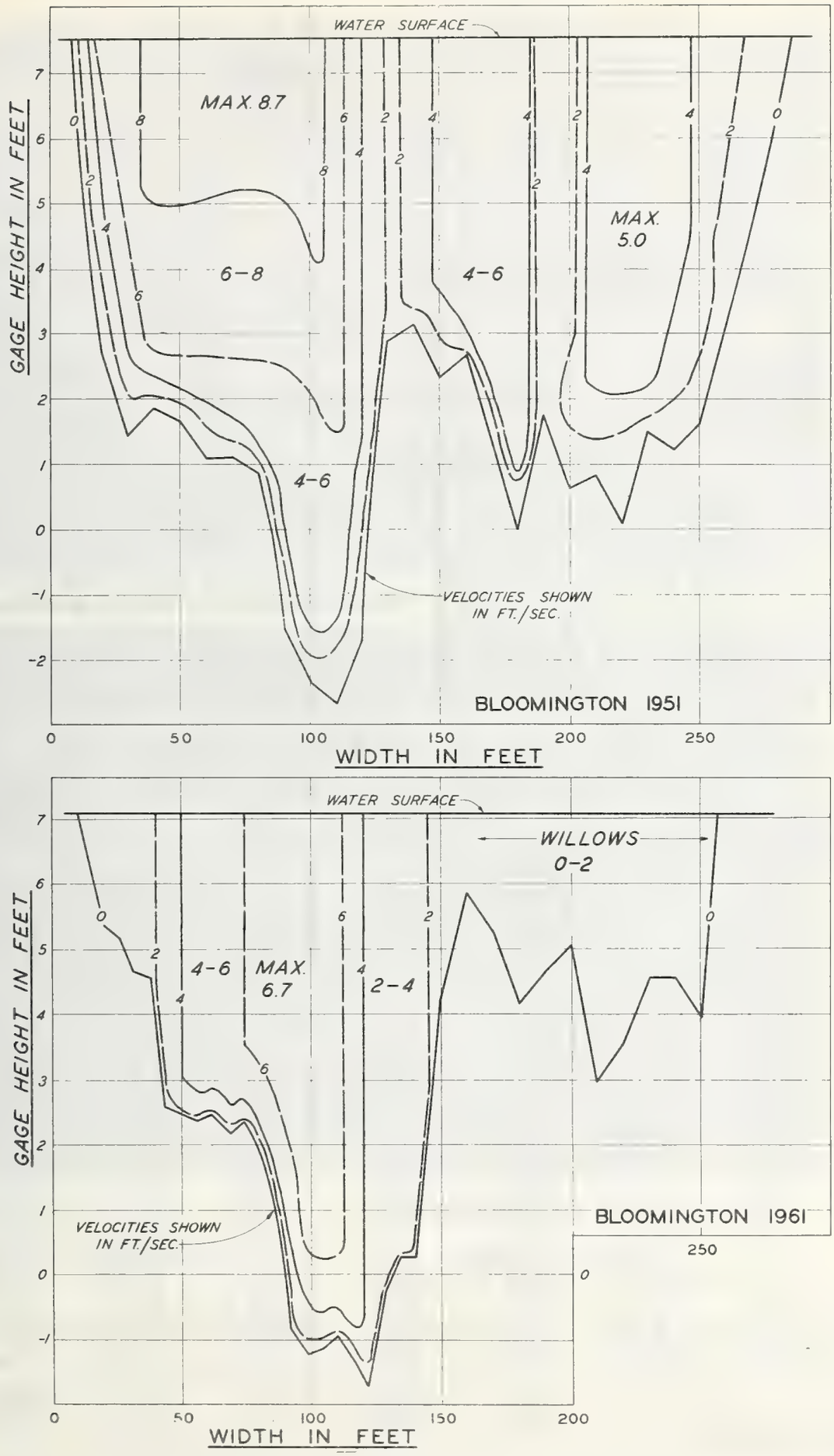


FIGURE 3. — Measured velocities of Republic River near Bloomington, 1951 and 1961.

TABLE 1.—*Summary of degradation range surveys below Harlan County Dam*[1]

Range No.	River miles (1961)	Miles below dam	Bankfull area[2]					Half-bankfull area				
			1951	1957	Change	1961	Change[3]	1951	1957	Change	1961	Change
			Sq. ft.	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*
1–2	237.9	1.5	5,027	5,365	+338			1,469	1,555	+86		
3–4	236.5	3.9	4,221	4,326	+105			1,587	1,629	+42		
5–6	235.2	4.2	3,248	3,150	−98			1,304	1,213	−91		
7–8	232.8	6.6	5,477	5,613	+136			1,832	1,824	−8		
9–10	231.7	7.7	3,234	2,770	−464			1,003	728	−275		
11–12	229.3	10.1	2,595	3,088	+493			1,645	1,776	+131		
13–14	227.2	12.2	3,339	3,508	+169	3,455	−53	1,512	1,746	+234	1,673	−70
15–16	224.6	14.8	3,720	3,531	−189	3,429	−102	1,859	1,646	−213	1,486	−160
17–18	221.9	17.5	3,533	4,166	+633	4,093	−73	659	1,053	+394	1,070	+17
19–20	215.0	24.4	3,077	2,864	−213			1,086	886	−200		
21–22	208.5	30.9	2,458	2,371	−87			800	717	−83		
23–24	190.6	48.8	3,508	3,115	−393			1,484	1,033	−451		
25–26	177.7	61.7	3,848	4,250	+402			1,283	1,520	+237		
27–28	160.1	79.3	2,176	2,568	+392			688	826	+138		
Total			49,461	50,685	1,224			18,211	18,152	−59		
Average			3,533	3,620	+87			1,301	1,297	−4.2		

[1] Dam at 239.4 river miles.
[2] Maximum depth used 12 feet.
[3] Change from 1957.

the main channel below Harlan County Dam. Prior to this time, the channel had remained essentially free of woody growth for many years, although these species have always grown in back channels and in many places near or above top of bank level. Little significance was attached to this new growth at the time, as it was assumed that the next flood would clean the channel. As it happened, a prolonged drought, depletion of streamflow by irrigation, and the initial filling of the two main stem reservoirs resulted in little or no flood activity along the

TABLE 2.—*Summary of degradation range surveys below Trenton Dam*[1] *keeping maximum depth down to 8.0 feet, which in several cases is below bankfull*

Range No.	River miles (1961)	Miles below dam	Bankfull area			Half-bankfull area		
			1949	1958	Change	1949	1958	Change
			Sq. ft.	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*
7	366.1	8.4	2,200	2,071	− 29	471	408	− 63
8	363.8	10.7	3,341	2,567	−774	1,086	599	−487
9	360.2	14.3	1,992	2,094	+102	457	680	+223
10	358.4	16.1	2,255	1,760	−495	1,022	549	−473
13	354.4	20.1	2,758	2,806	+ 48	1,260	1,248	− 12
13a	353.9	20.6	1,880	1,770	−110	1,022	633	−389
14	348.8	25.7	1,602	1,498	−104	593	500	− 93
14a	347.1	27.4	1,492	1,380	−112	437	380	− 57
15	345.6	30.9	2,060	1,790	−270	548	420	−128
16	341.9	32.6	2,381	2,608	+227	578	612	+ 34
17	334.7	39.8	2,811	2,788	− 23	1,160	1,165	+ 5
18	330.5	44.0	3,259	2,560	−699	900	480	−420
20	323.9	50.6	3,127	2,605	−522	1,305	775	−530
22	317.7	56.8	2,596	2,414	−182	958	501	−457
23	313.6	60.9	2,882	2,694	−188	1,230	1,010	−220
23a	312.6	61.9	2,958	3,040	+ 82	785	866	+ 81
24	312.1	62.4	2,734	2,136	−598	1,080	625	−455
25	310.7	63.8	3,250	2,985	−265	1,345	1,087	−258
26	308.1	66.4	3,970	3,185	−785	1,355	1,100	−255
27	305.7	68.8	3,110	2,880	−230	1,225	1,045	−180
28	301.9	72.6	4,820	5,170	+350	1,990	2,110	+120
28a	298.0	76.5	3,973	3,753	−220	1,695	1,660	− 35
29	296.2	78.3	3,554	2,134	−1,420	1,240	510	−730
31	290.1	84.4	3,140	4,360	+1,220	1,233	1,380	+147
32	284.6	90.1	3,280	3,830	+550	1,595	1,975	+380
34	276.7	97.8	2,810	2,133	−677	920	706	−214
Total			74,235	69,011	−5,124	27,490	23,024	−4,466
Average			2,855	2,654	− 197	1,057	886	− 172

[1] Dam at 374.5 river miles.

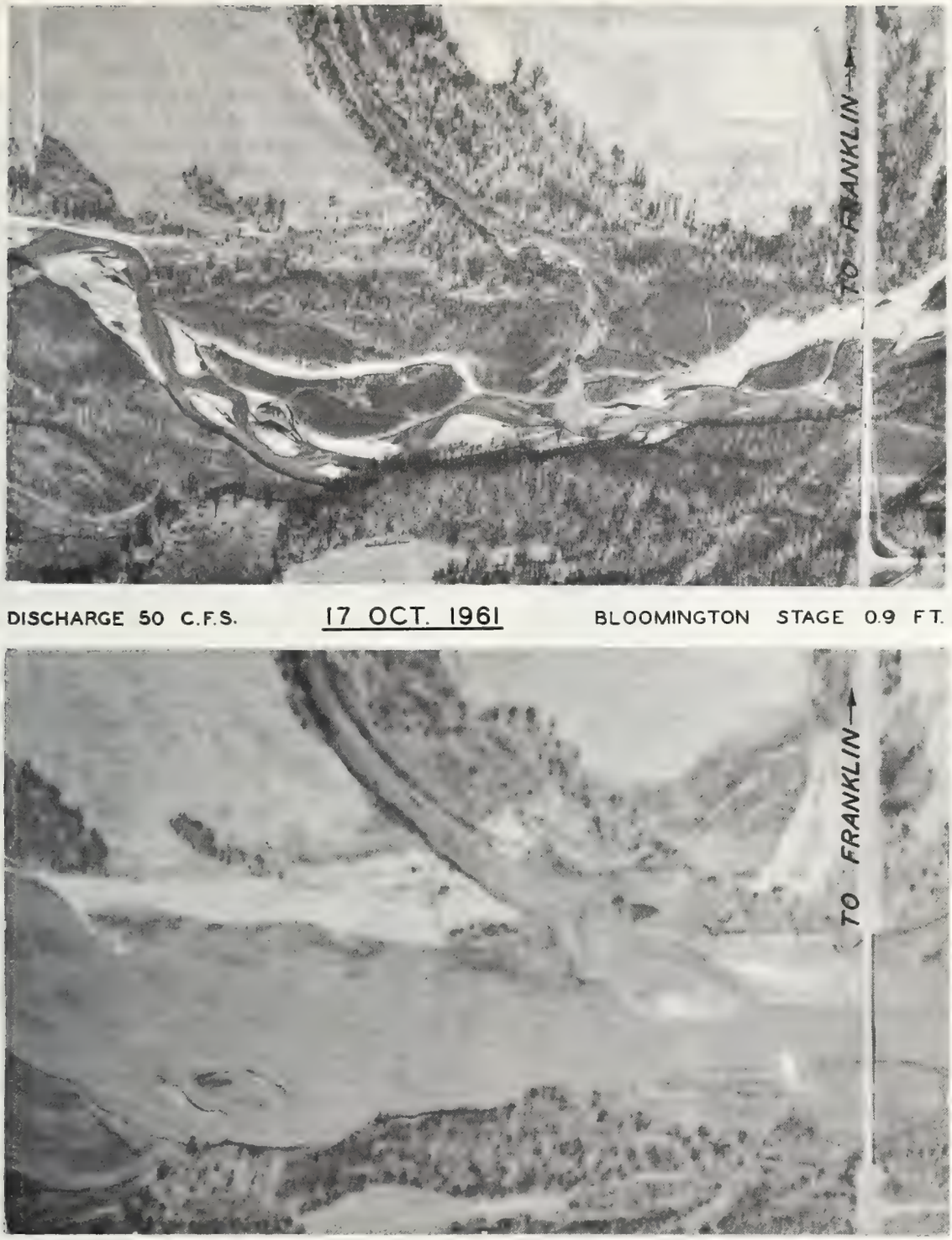


FIGURE 4. — Willow encroachment in the Republican River, 1951 and 1961.

river to the Kansas-Nebraska State line until 1957. By 1955, local residents expressed concern about the willow growths blocking the channel. At that time the growth had reached a height of 4 to 7 feet and the area of younger growth had extended from Trenton Dam to the downstream reaches in Kansas. By 1957, the older growth was 7 to 15 feet high and accretions of sand up to 3 feet in depth had formed in the growths thus increasing the height of the islands and bars. Much of the channel area is now covered by willows and cottonwoods of a size that could be dislodged only by the most extreme floods or by mechanical means. Deposit of sand in the larger growth has increased to 5 feet in depth at some locations. Fortunately, aerial pictures of the channel below Harlan County Dam were made in 1951, and between Trenton Dam and Harlan County Reservoir in 1958. In 1961, the reach below Harlan County Dam was photographed again. Measurements were made of the area of growth between high banks. Figure 4 shows a typical reach of channel in 1951 and how it has been changed by 1961. Figure 5 shows that 40 to 60 percent of the 1951 channel area has been occupied for a major part of its length by woody growth.

Bank Erosion

Comparisons of the 1951 and 1961 aerial photographs between Harlan County Dam and Clay Center show that the willow growth has had the overall beneficial effect of reducing bank erosion from 121 miles to 20 miles. This reduction in the extent of caving banks is attributed principally to the growth that was established on the banks during the 1950 drought period. Reduced current velocities due to growth in the channel have also been beneficial from this viewpoint, except in instances where deposits in the growth have increased velocities next to the banks. Instances may be found where the concave (eroding) side of river banks have filled in with willows and sand accretion. In tentatively establishing the outside limits for channel restoration purposes, a deviation from the 1951 channel has been made, when possible, to take advantage of the protective cover provided by recent willow growth. At many critical locations, local interests have successfully used

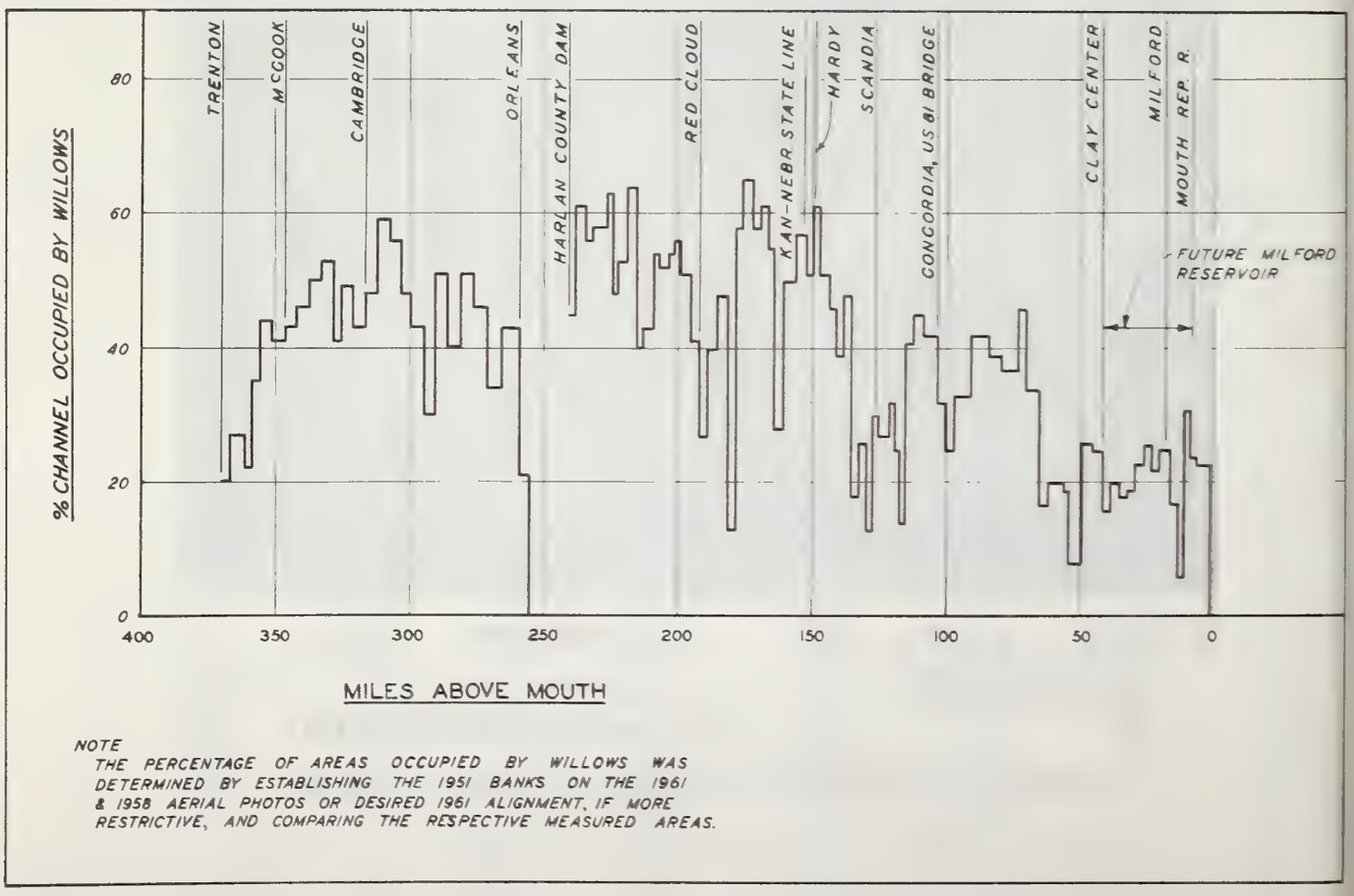


FIGURE 5. — Percentage of channel occupied by willows in Republican River.

old car bodies and other types of improvised light revetment to prevent further erosion and induce sedimentation. The areas of active bank erosion that remain will present a problem so long as corrective measures are not taken. A substantial part of seriously eroding bank areas can be greatly improved or eliminated by completing cutoffs that can be made at comparatively low cost. Some areas would be improved by removal of woody growth from the main channel. It is anticipated that light revetment such as car bodies, and continued encouragement of bank growth will be required in addition to cutoffs and channel clearing to completely eliminate undesirable caving banks.

Hydrologic History

Records

Fragmentary records of stream discharge of the Republican River are available dating back to 1894. Several gaging stations were established in the 1920's and good records are available since the mid-thirties. Major floods have often washed out gage installations, and actual current meter measurements of floods much in excess of flood stage were virtually impossible prior to construction of high bridges in the last 20 years. Although the quality of records has improved as access to stations during floods became possible, due allowances must be made in considering changes in channel capacity.

Flood Peaks and Runoff

The erratic hydrologic history of the Republican River is illustrated by the annual maximum discharges and annual runoff values given on figure 6 for the station at Bloomington. From the beginning of the record in 1929 through 1951, more than half of the peak annual maximum discharges at this station were in the 10,000 c.f.s. range, 4 years exceeded 15,000 c.f.s., and 2 years, 1935 and 1947, were 260,000 c.f.s. and 140,000 c.f.s., respectively. With a few exceptions, the annual runoff volumes varied about as the peak values. There was an abrupt break in the pattern after 1952 when the peak discharge varied from 1,000 to 5,000 c.f.s. Low discharges after this time were the result of Harlan County Reservoir regulation following initial filling of the conservation pool in 1957. The record low annual runoff volumes beginning with water year 1953 and extending through 1956 is explained by sustained drought conditions during this period.

Channel Capacities

The channel capacity that can be related to the gage at Bloomington is roughly approximated by the top line on figure 6 for the present state of valley development. Comparison with the peak discharges and runoff volumes also shown in the figure indicate a rough correlation with channel capacity. There was a gradual improvement in channel capacity from the 4,000 to 5,000 c.f.s. range prior to the 1935 flood to 16,000 c.f.s. after the 1947 flood. Subsequently, there was a comparatively rapid deterioration to 4,000 c.f.s. in 1957. A small, but discernible, improvement is indicated in the last few years, which had near normal volumes of flow. During the study of Republican River channel conditions, similar diagrams were prepared for other stations. The improvement between 1935 and 1947 is not so regular and pronounced at some of the other stations, but the deterioration between 1952 and 1957 is evident. A particular exception was noted for the reach below Trenton Dam where the improvement since 1960 was lacking. In fact, deterioration is still in progress at Cambridge about midway between Trenton Dam and Harlan County Reservoir. Analysis of records from 1917 to 1934 at Culbertson (near Trenton Dam) and Wakefield (near Clay Center) indicate that there had been a fairly stable channel during this earlier period. One investigator found evidence that a flood which may have been comparable to the 1935 flood occurred in 1826. If channel scour comparable to 1935 condition occurred in 1826, the 1917-to-1934 stabilized channel, with 4,000 to 5,000 c.f.s. capacity, would be indication of what could be expected to develop eventually without reservoirs to change the flow regimen.

Flow Duration

Duration studies were made for all stations with substantial records. Curves for three significant periods of record, 1934-52, 1953-56, and 1957-61, are shown in figure 7 for the station at Bloomington. These curves confirm the extremely low flows during the drought period 1953-56, and show that floods since 1956 are somewhat less than the normal experienced prior to 1952. The analysis of flows below Trenton Dam indicated the same tendency during the drought period in the 1950's. Whereas the drought period was also evident at the lowest station, Clay Center, the variation from normal was not as pronounced.

Pool Operation

Impoundment of flow in the conservation pools of several headwater reservoirs began about 1950. Harlan County Reservoir was placed in operation for conservation storage in October 1952, and Trenton Dam (Swanson Lake) began conservation operation late in the calendar year 1953. Lovewell, a small tribu-

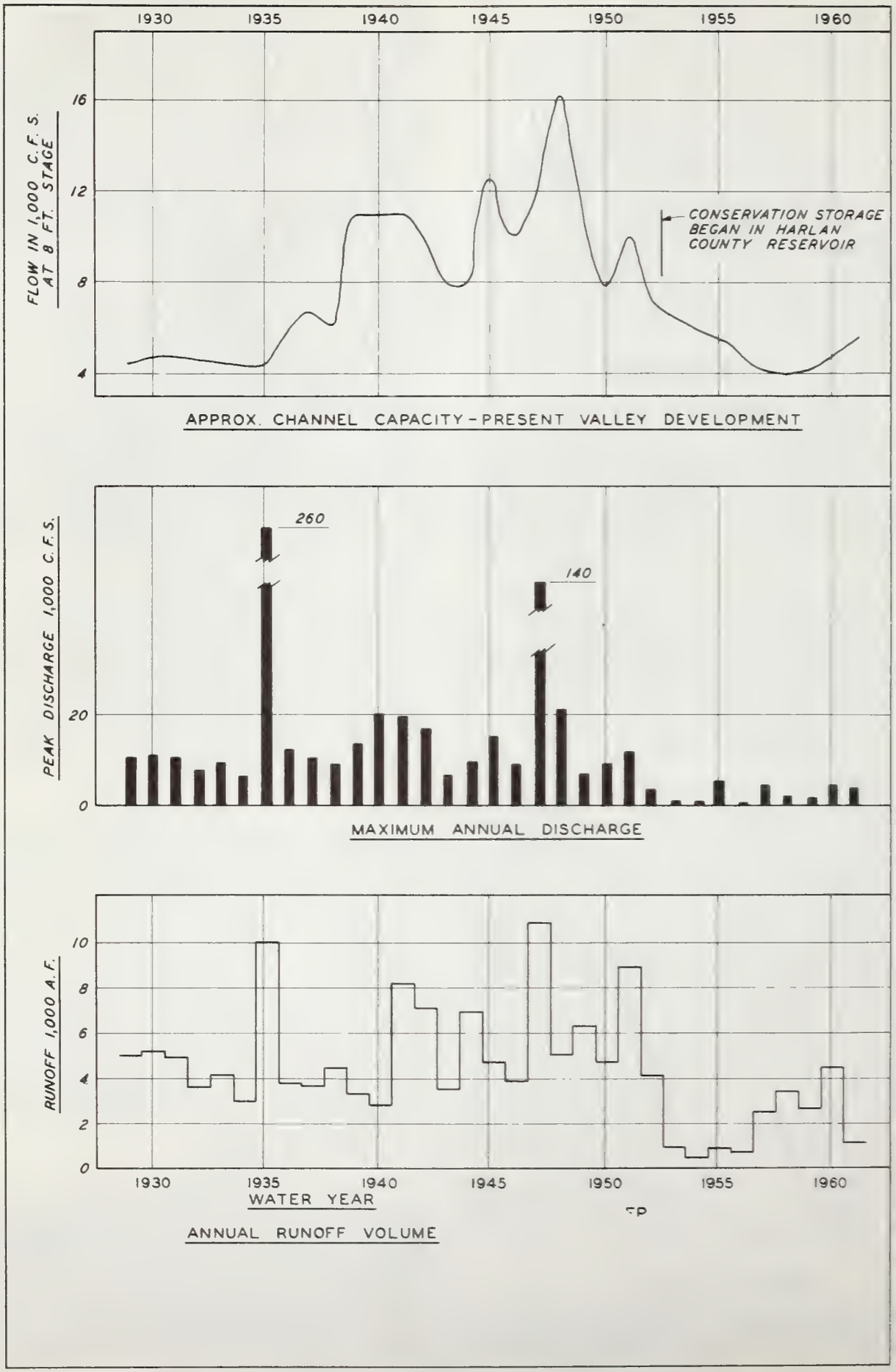


FIGURE 6. — Hydrologic history of Republican River at Bloomington, 1930-60.

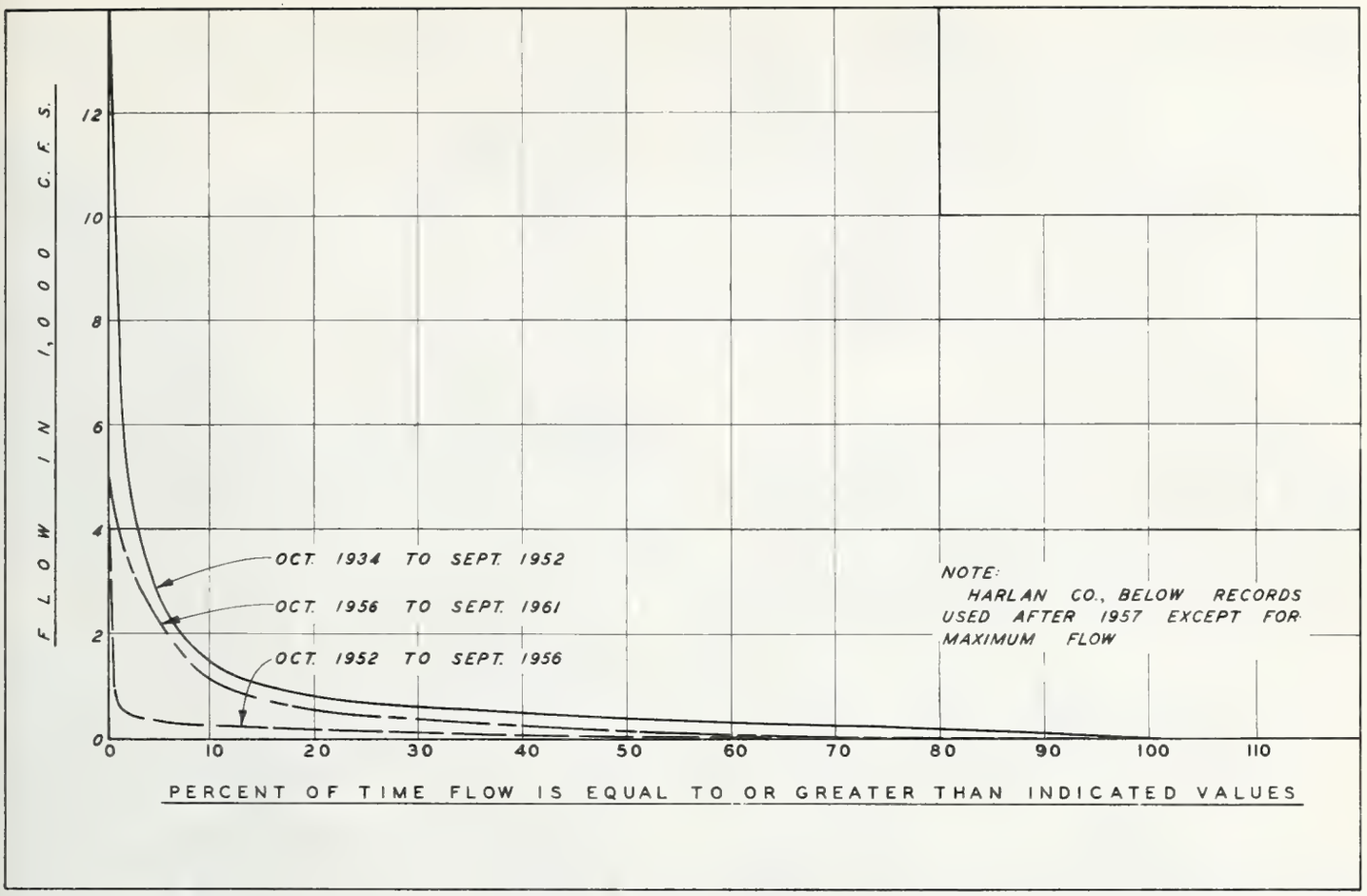


FIGURE 7. — Durations of flow below Harlan County Dam at Bloomington.

tary reservoir in the lower basin, began impoundment in 1957, and Milford Reservoir, now under construction, is scheduled to begin impoundment of conservation storage in 1966. Initial filling of Harlan County Reservoir and Swanson Lake was completed in the spring of 1957. Releases to the channel up to about 350 c.f.s. were made at both projects for irrigation during initial filling. Beginning in 1957, reservoir releases have been limited to about 2,500 c.f.s. at Trenton Dam and 4,000 c.f.s. at Harlan County Dam, due to reduced downstream channel capacities. Usually the channels are not used to full capacity, as some margin of safety is desirable for unpredictable local inflow, and some uncertainty usually exists as to the exact capacity that should be used. Substantially more volume of flood storage has been available to release from the Harlan County Reservoir since 1957 than from the Trenton Dam. Preliminary studies indicate a channel capacity of 7,000 c.f.s. and 10,000 c.f.s., respectively, below Trenton Dam and Harlan County Dam would be desirable for effective regulation of the Republican River reservoir system. Capacity, in addition to these values, would, of course, be desirable if a practicable method can be devised to make it available.

Sediment

Channel Characteristics

The Republican River is a naturally meandering sand-bottom stream with slopes gradually increasing from 3½ feet per mile at the mouth to 9 feet per mile at Trenton Dam. The river valley averages about 1 mile in width and is formed generally of sandy soil that is easily eroded. Streambanks are usually well defined and average about 8 feet in height between Trenton Dam and Harlan County Dam and about 12 feet in height downstream from Harlan County Dam. Banks well below the average, with adjacent low bottom land, are found at irregular intervals. The banks are prone to erode on the outside of bends when not protected by natural vegetation or by artificial measures. The channel has made many natural cutoffs, usually during extreme high water. The back channels normally are soon occupied by a growth of willows and cottonwoods, which accelerate the entrapment of sediment and closure of the channels to normal river flows. A few

natural islands are to be found, but for the most part all inbank flow is carried in one channel. Bottom widths average 600 feet below Trenton Dam and 450 feet between Harlan County Dam and the mouth. The sand bed is shaped into islands and shore bars that fluctuate in size and location from one flood to the next when not held in place by vegetation. Woody growth and related sediment accumulations on low islands and bars, as described previously, has impaired bankfull channel capacity and has in some instances accelerated caving banks in the remaining active areas.

Sediment Data

Sediment data have been collected on the main stem of the Republican River at eight locations. These data have been used to estimate sediment loads shown in table 3.

Streambed Material

Early in 1961, about 150 surface samples of the streambed material were collected along 200 miles of the river channel below Harlan County Dam. The samples were divided equally between (1) the low-water channel, (2) bars not exceeding 2 feet above low water, and (3) bars 2 to 5 feet above low water. Lenses of silt, which constitute roughly a fifth of the material in islands and bars built up by sediment deposits, are usually found at lower levels and are not reflected in the surface samples. As shown in table 4, the surface of the high bars and islands contain a significantly higher percentage of the fine sands. The bed of the low-flow channel was probed with a ½-inch rod 7 feet in length. The bottom of the sand was not reached except in some instances where the channel was adjacent to the hills that define the alluvial plain. In these instances penetration was limited to 4 or 5 feet. It appears from field observations that bed material between Trenton and Harlan County Dams is of a similar nature.

TABLE 3.—*Sediment load of Republican River at selected stations*

Station	Average annual load in tons			
	Before reservoirs		With reservoirs[2]	
	Suspended sediment	Bedload[1]	Suspended sediment	Bedload[1]
Trenton, Nebr.	1,230,000	.....	15,000	.....
Orleans, Nebr.	5,600,000	450,000	920,000	310,000
Harlan County Dam, Nebr.	.....	.....	17,000	.....
Bloomington, Nebr.	5,700,000	475,000	60,000	160,000
Guide Rock, Nebr.	.....	.....	390,000	200,000
Hardy, Nebr.	.....	.....	725,000	.....
Concordia, Kans.	.....	.....	1,420,000	.....
Clay Center, Kans.	8,323,000	500,000	2,580,000	380,000

[1]Estimated by Straub's equation in Missouri River 308 Report.

[2]These estimates are based on normal runoff below the dams and on trap efficiency of near 100 percent.

High Bar and Island Composition

Typical high bars and islands formed by sediment deposits in vegetative growth were sampled to the low-flow elevations at 10 locations, and sections exposed by erosion were observed at other places. The samples were found to be composed of sand similar to the surface samples previously described, except layers of silt were found at random locations in the vertical section. Most islands and many of the bars are protected by stranded driftwood that will require removal if a willow-clearing program is to be effective. Sieve analyses of the samples are summarized in table 5.

It was found expedient to sample and analyze the sand and silt layers separately. A field estimate indicates the silt layers constitute about 20 percent of the high bar and island material. The weighted average size-distribution shown above was based on this observation. While there were variations from place to place, no definite trend in size-distribution was established in relation to river mileage or other iden-

TABLE 4.—*Summary of Republican riverbed material samples, 1961*

Location of samples	Percentage of material retained by sieve sizes								
	¾ (20 mm.)	⅜ (10 mm.)	4 (4.8 mm.)	10 (2 mm.)	18 (1 mm.)	40 (0.44 mm.)	70 (0.22 mm.)	200 (0.070 mm.)	Less than 0.005 mm.
Low flow channel:									
Increments	0	0.4	1.9	7.1	14.5	46.7	25.6	3.8	0
Summation[1]	0	.4	2.3	9.4	23.9	70.6	96.2	100.0	100.0
Low bars:									
Increments	.1	.4	1.5	6.3	12.9	40.5	32.0	4.9	0.4
Summation[1]	.1	.5	2.0	8.3	21.2	61.7	93.7	98.6	100.0
High bars:									
Increments	.2	.1	0.4	2.0	4.5	30.0	44.0	18.6	0.2
Summation[1]	.2	.3	.7	2.7	7.2	37.2	81.2	99.8	100.0
Average:									
Increment	.1	.4	1.3	5.2	10.6	39.2	33.9	9.1	0.2
Summation[1]	.1	.5	1.8	7.0	17.6	56.8	90.7	99.8	100.0

[1]Summation is percent coarser.

TABLE 5.—*Particle-size distribution of sediment deposits of Republican River*

Sample	Percentage of materials retained								
		Sand					Silt		Clay
	Fine gravel (4.8 mm.)	Very coarse (2.0 mm.)	Coarse (1.0 mm.)	Medium (0.44 mm.)	Fine (0.22 mm.)	Very fine (0.075 mm.)	(0.05 mm.)	(0.10 mm.)	(0.005 mm.)
Average of sand samples:									
Increments	2.0	2.0	10.0	37.0	32.0	9.0	6.0	1.0	1.0
Summation[1]	2.0	4.0	14.0	51.0	83.0	92.0	93.0	99.0	100.0
Average of silt samples:									
Increments	.....	.....	.....	2.0	17.0	16.0	47.0	5.0	13.0
Summation[1]	.....	.....	.....	2.0	19.0	35.0	82.0	87.0	100.0
Weighted average: Sand, 80 percent Silt, 20 percent	.....	2.0	10.0	30.0	29.0	10.0	14.0	2.0	3.0

[1]Summation is percent coarser.

tifiable features. Schematic sections of a typical island are given on figure 8.

Willow Root System

Since the composition of the root system has considerable significance in determining methods that might be adopted for removal of objectionable islands and bars, a study was made to determine its nature. The principal woody

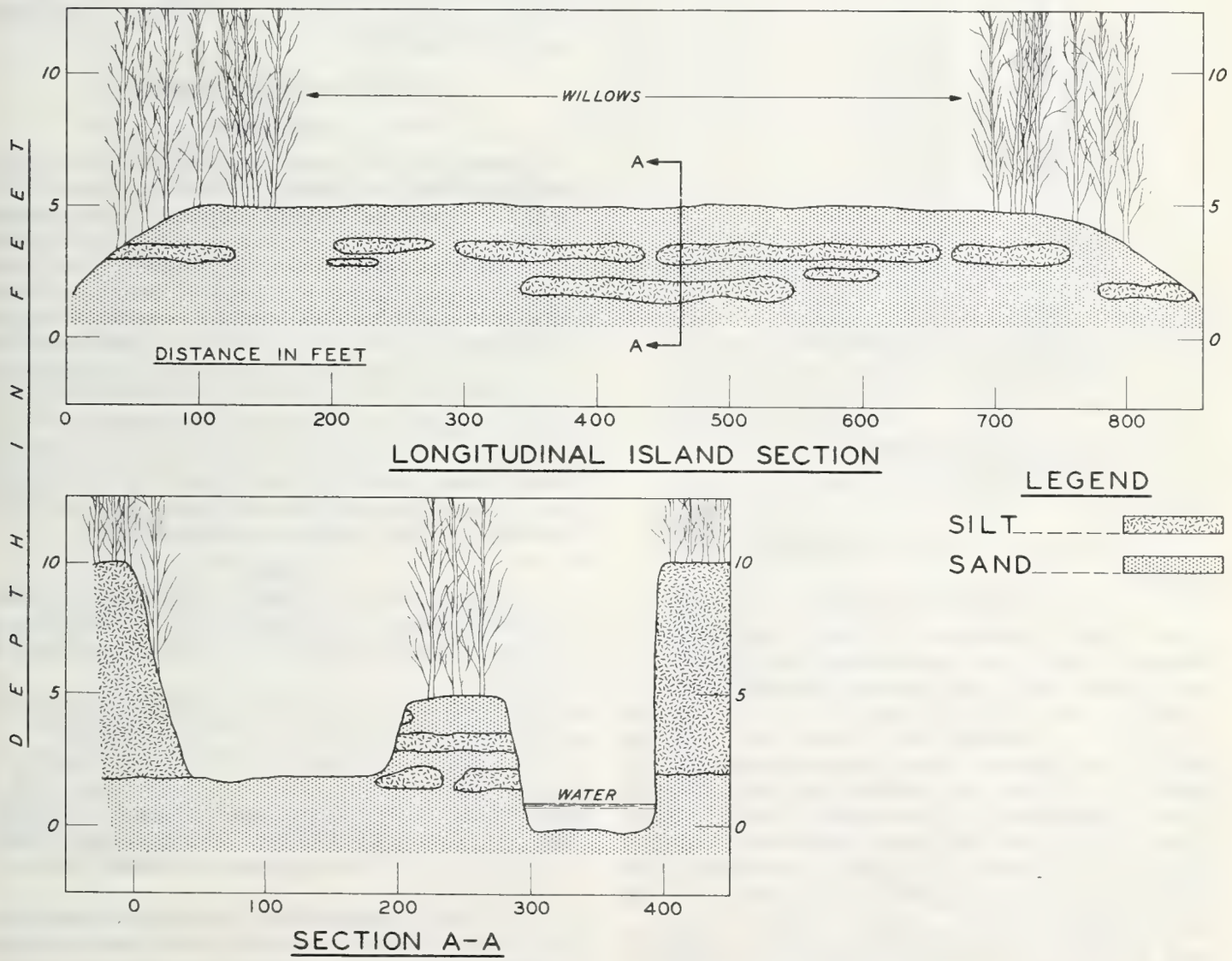


FIGURE 8. — Schematic sections of typical island in Republican River.

growth on the islands was found to be willows. The sides of selected islands were excavated from top to bottom to expose the root systems. The entire volume between low water and top of island was well-filled with roots varying from threadlike size to a half-inch in diameter. Many of the willows apparently started just above low water and the center stem extends upward to the surface with roots branching off at all levels, while others obviously started at higher levels and have extended small roots down to low water. Figure 9, showing typical root systems, was prepared from photographs of the exposed roots.

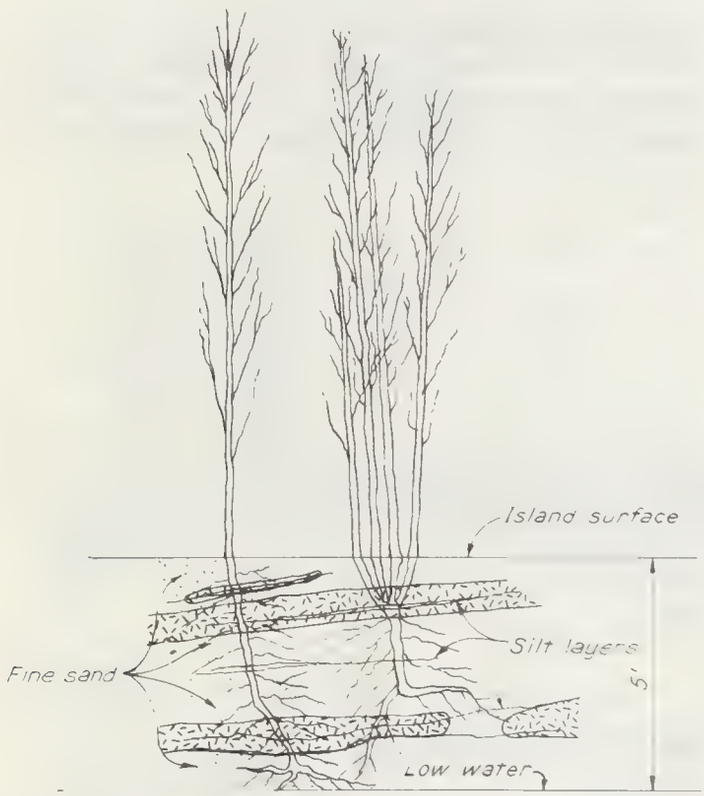


FIGURE 9. — Willow root development.

Roughness Coefficients

A typical 5½-mile reach of the channel below Harlan County Dam, extending from Bloomington downstream, was selected for field tests to evaluate channel-roughness factors, including the effect of willows that cover many of the islands and bars. The channel between high banks was about 60 percent occupied by willows. Occasional cottonwoods were noted. Twenty cross-sections were established, three of which were located at degradation ranges. Discharge measurements were made at six of the sections for regulated releases up to 3,500 c.f.s. Analysis of the discharge measurements and related stream slopes indicated average "n" values of 0.20 and 0.036 for willow-covered bars and for sand, respectively. The results of these tests were used to compute a rating curve at Bloomington for both actual conditions and with willows removed from the main channel, as shown on figure 10. This computation method does not include an allowance for the probable increased efficiency due to better flow configuration with the willows removed and the sand bed smoothed out by floodflows. It was not practicable to extend the tests to evaluate this factor.

The increase in channel capacity resulting from willow removal is dependent on willow area, channel area, and channel capacity. It appears reasonable to assume that the ratio of the percentage of increase of capacity in a reach resulting from willow removal to the percentage of willows in the reach is constant. On this basis, the results in the test reach can be applied to other reaches and the effects of willow removal for the entire Republican River can be evaluated.

Engineering Studies

General

Serious concern over Republican River channel deterioration began in the spring of 1957 when the reduced channel capacities became apparent. Several reconnaissance-type field studies and analyses of available office data were made during the period 1957-60 to evaluate the situation as it was related to reservoir regulation requirements. In December 1960 the problem was called to the attention of the Missouri River Division Board of Sediment Consultants [1] and suggestions of the board members were followed in the collection of field data and the analysis of available information, with a view to formulating recommendations for corrective action, if such should be indicated.

Findings

Principal factors related to the loss of channel capacity and possible future restoration as indicated by the studies are summarized as follows:

A. The loss of channel capacity appears to have been caused by three principal factors:

(1) Extensive use of former low wasteland since 1951 for farm crops from Trenton Dam to the mouth of the Republican River has lowered the maximum upper level of regulated release over 1 foot and accounted for roughly 50 percent of the loss.

(2) Partial blocking of the main channel, disruption of the flow pattern by permanent woody growth on former low bars and islands, and de-

[1] The Missouri River Division of Sediment Consultants Board is composed of H. A. Einstein, L. G. Straub, and Vito Vanoni.

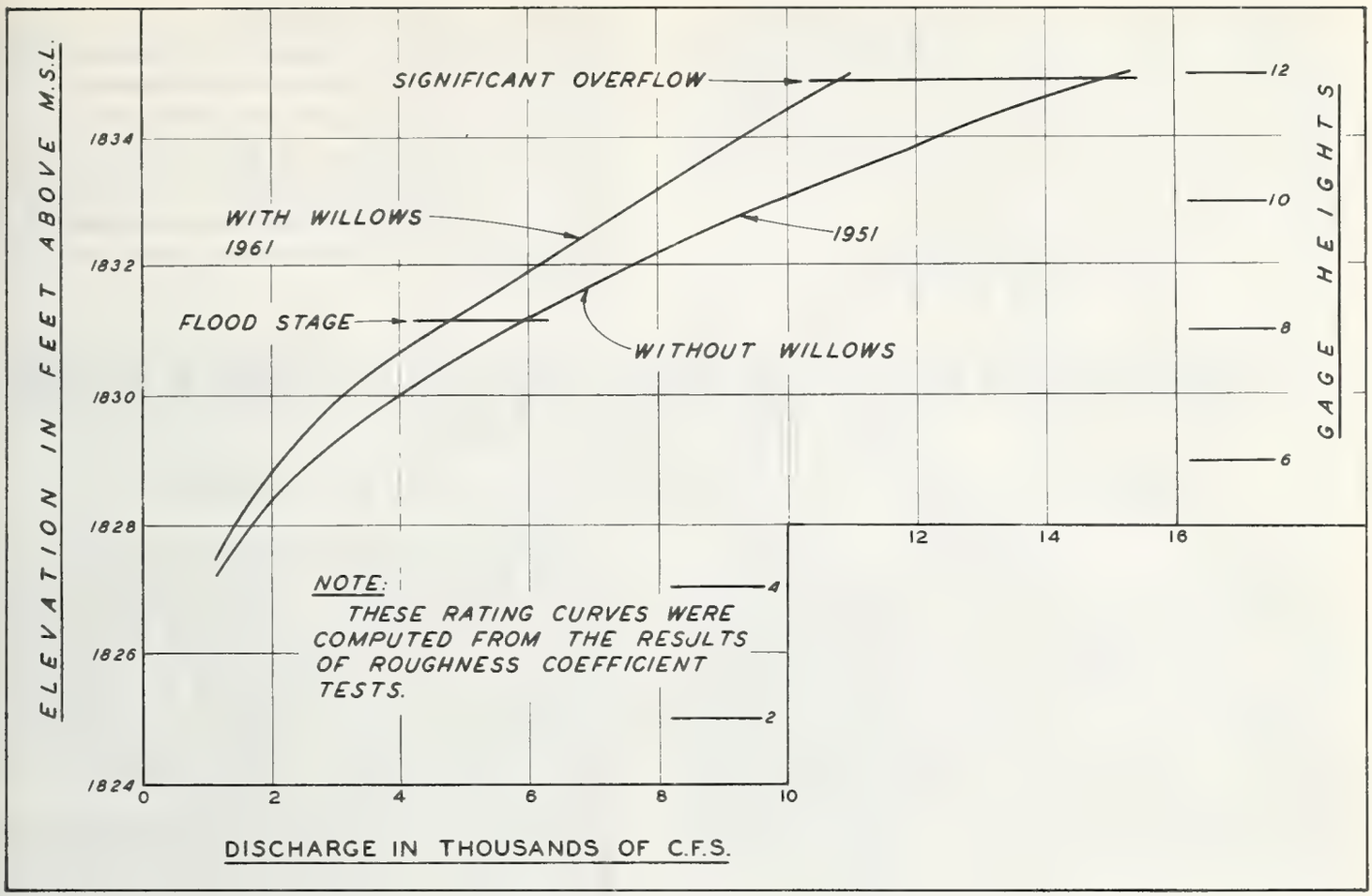


FIGURE 10. — Republican River stage-discharge relations near Bloomington, 1951 and 1961.

posits of bed material up to 5 feet in depth in the woody growth have accounted for the remaining loss of channel capacity.

(3) Initial filling and subsequent regulation of the Swanson Lake (Trenton Dam) and Harlan County Reservoir for flood control have eliminated or greatly reduced the natural flood peaks that served to flush the channel and in that way has contributed, to an undetermined extent, to the loss due to woody growth and deposits on the islands and bars.

B. There is evidence that the former enlargement of channel capacity to 15,000 c.f.s. or more below Harlan County Dam was due to a combination of the 1935 flood and the succeeding 12 years of a sustained wet cycle. It is likely that a repetition of the 1826-1934 climatic and streamflow cycle would have caused the channel to assume the same 4,000 to 5,000 c.f.s. capacity available before the 1935 flood. Essentially the same reasoning applied to the channel below Trenton Dam.

C. The grain-size of the sandbars and islands, the root structure of the willows, and the configuration of the bars and islands indicate that the channel would be readily eroded to former natural conditions by bankfull streamflow after clearing of the woody growth, decay of the roots, and the removal, where present, of snags or resistant obstructions at the heads of the islands and bars.

D. Much of the channel capacity, which was lost during the past decade, could be restored by removal of the woody growth and maintenance of the denuded areas.

E. Removal of woody growth from bars and islands between the high banks and completion of the more obvious cutoffs will alleviate existing caving banks to the extent that remedial measures can be taken with light treatment, such as plantings and dumping of car bodies.

Conclusion

Hydrologic data, sediment information, and aerial mapping accumulated prior to recognition of the channel deterioration proved invaluable. These data combined with the results of a short period of intensive field work made possible a fruitful engineering study and evaluation of the problem. This experience again emphasized the advisablity of analyzing in advance the potential channel changes that may be

expected when the flow regimen is changed. As a result of the studies, it was possible to recommend corrective measures in this particular situation. It is anticipated that the results of the study also will be of value in planning and administering water-control programs in the surrounding areas with similar channel, flow, and climatic characteristics.

Acknowledgments

Much of the material incorporated into this paper was prepared originally for the Republican River Review Report which is nearing completion in the Kansas City District. Many individuals in the Planning and Reports Branch, under the direction of Willis G. Fish, contributed to the studies. The writer is especially indebted to Alexander M. Gow and Charles A. Burgtorf for review of the text and many valuable suggestions.

DEGRADATION STUDY OF THE MIDDLE MISSISSIPPI RIVER, VICINITY OF ST. LOUIS

[Paper No. 48]

By THOMAS F. MAHER, *hydraulic engineer, U.S. Army Engineer District, St. Louis*

Synopsis

The results of a study of the relation in time between hydrographic surveys and stage and discharge observations in the vicinity of St. Louis, Mo., are presented. An attempt has been made to include as much basic data as are obtainable. Data for the analysis were obtained from the files of the U.S. Army Engineer District, St. Louis. Analysis of results shows accelerated degradation was brought about by works of man.

Degradation Study

St. Louis, Mo., lies on the right, or west, bank of the Mississippi just downstream of the confluence of the Missouri River. The drainage area above St. Louis is divided into two major river basins, the Missouri and upper Mississippi, which have a combined area slightly in excess of 700,000 square miles. Approximately 530,000 square miles lie in the Missouri River basin and the remaining 170,000 square miles in the upper Mississippi River basin.

The flood seasons in the Missouri and upper Mississippi basins occur during the period, March to July. Except for periods of drought, the low water season is generally in the winter months, December through February.

The mean discharge of the Missouri River is about 82,000 cubic feet a second with an estimated maximum of about 900,000 in 1844 and a minimum of about 4,000 in 1940. The mean discharge of the upper Mississippi River is about 95,000 cubic feet a second with an estimated maximum of 565,000 in 1851 and 1858. Minimum discharge occurred in 1948 with a discharge of about 8,000 cubic feet a second. The mean discharge at St. Louis, Mo., is about 175,000, with an estimated maximum of about 1,300,000 in 1844 and a minimum of about 20,000 in 1933.

Fragmentary and intermittent river stage records were kept in St. Louis, Mo., between 1837 and 1860. Complete gage records are available for the year of 1844, when the greatest flood of record occurred. The first continuous records of stage were initiated in 1861, when the present Market Street gage was established at mile 179.6 above the mouth of the Ohio River. Stage and flow records stated herein are referred to the Market Street gage.

The earliest streamflow measurements taken at St. Louis were made by the city engineer of St. Louis in 1866. However, flow measurements were estimated as early as 1837. Since 1866, streamflow measurements have been made by the city of St. Louis, Mississippi River Commission, U.S. Corps of Engineers, and the U.S. Geological Survey. Although discharge measurements have been taken at several localities in the area, the results are all referred to the Market Street gage.

Hydrographic surveys covering parts of the area under discussion, from which it is possible to chart the changes in bed and bank configuration, are available since 1837. The first comprehensive survey was made by Lt. Robert E. Lee in connection with his work on the St. Louis Harbor.

Data from historic maps reveal that the Mississippi River at Vine Street in St. Louis was 3,100 feet wide in 1803. Due to shifting and cutting of the Illinois channel currents eastward of Bloody Island, the river gradually enlarged to 3,200 feet in 1808, 3,700 feet in 1837, 3,900 feet in 1843, and reached a maximum of 4,200 feet in 1849. Because of the large commerce resulting from the steamboats of the era, it became necessary to preserve the channel in front of the city of St. Louis, which was deteriorating rapidly. The magnitude of the interests of this wealthy and rapidly growing city demanded that the river, throughout the

entire length of the 13 miles of the city river front, be made to flow in a permanent channel having stable banks. To effect this, the city and private corporations began work in 1838 on a series of dikes from the Illinois shore, intended to confine the river in a definite channel. The Federal Government recognized the magnitude of the work and initiated work based upon the plan proposed by Lt. Robert E. Lee. Limit lines finally adopted for the St. Louis Harbor were 1,500 feet at low water (L.W. 1863 = zero on gage) and 2,000 feet at bankfull (30 feet on gage). The river at St. Louis as it existed in 1837 is shown on figure 1. On figure 1 is also shown the river at the present time in dashed lines. It will be noted by comparison the contraction from about 4,000- to 2,000-foot width.

In order to show the change in channel regimen resulting from the contraction work, comparative cross sections are shown on figure 2. The cross sections are at approximate mile 180 just downstream at Eads Bridge. For the earlier sections the deeper water is on the Illinois side, whereas after completion of the contraction works in about 1875, there is a gradual and marked deepening of the Missouri channel. The Missouri channel was at elevation 380+ in 1837 and is at elevation 360— in 1963. To illustrate this bed lowering further, channel thalweg profiles are plotted and shown on figure 3. The channel profiles extend from about mile 175 to mile 185 and show the overall lowering of the bed from 1837 to date.

Individual cross sections or channel profiles are not always conclusive as to what is happening in a channel, because the surveys performed may not be phased with the phenomenon. Discharge measurements over a long period, referred to a permanent gage, will portray changes in channel regimen. On figure 4 is shown the stage for a constant discharge of 40,000 cubic feet a second from 1837 to date. In 1837, 40,000 cubic feet a second passed the St. Louis-Market Street gage at a plus 6-foot stage, whereas in 1963 this same discharge passes at stage of minus 6-foot. This is a lowering of 12 feet in about 120 years. The rate of lowering has been almost constant since 1875. Table 1 shows a list of selected discharge measurements.

It is not the intention of this paper to claim that there is no filling of the channel due to bedload movement. The purpose is to show the general long term trend. There have been numerous occasions in the past when the bed elevation has fluctuated markedly, the cause being attributed to hydrologic conditions in the upstream basins. One documented fluctuation occurred in the winter of 1951-52 after the great Kansas River basin flood of 1951. This flood carried tremendous amounts of bedload material, which was deposited not only in the channel but in the extensive overbank areas below the junction of the Kansas and Missouri Rivers. Due to the flatter slopes of the Mississippi in relation to the Missouri, large channel depositions occurred in the Mississippi below

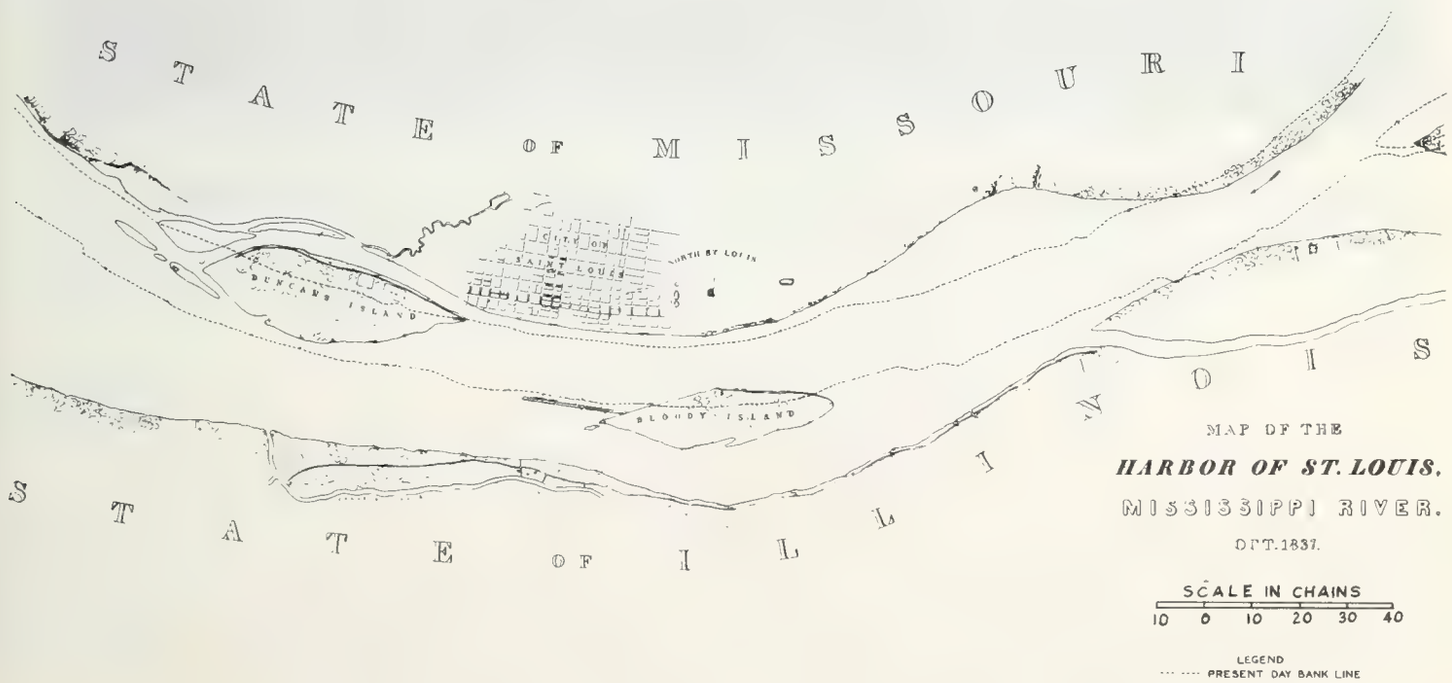


FIGURE 1. — Present-day and 1837 channel configurations.

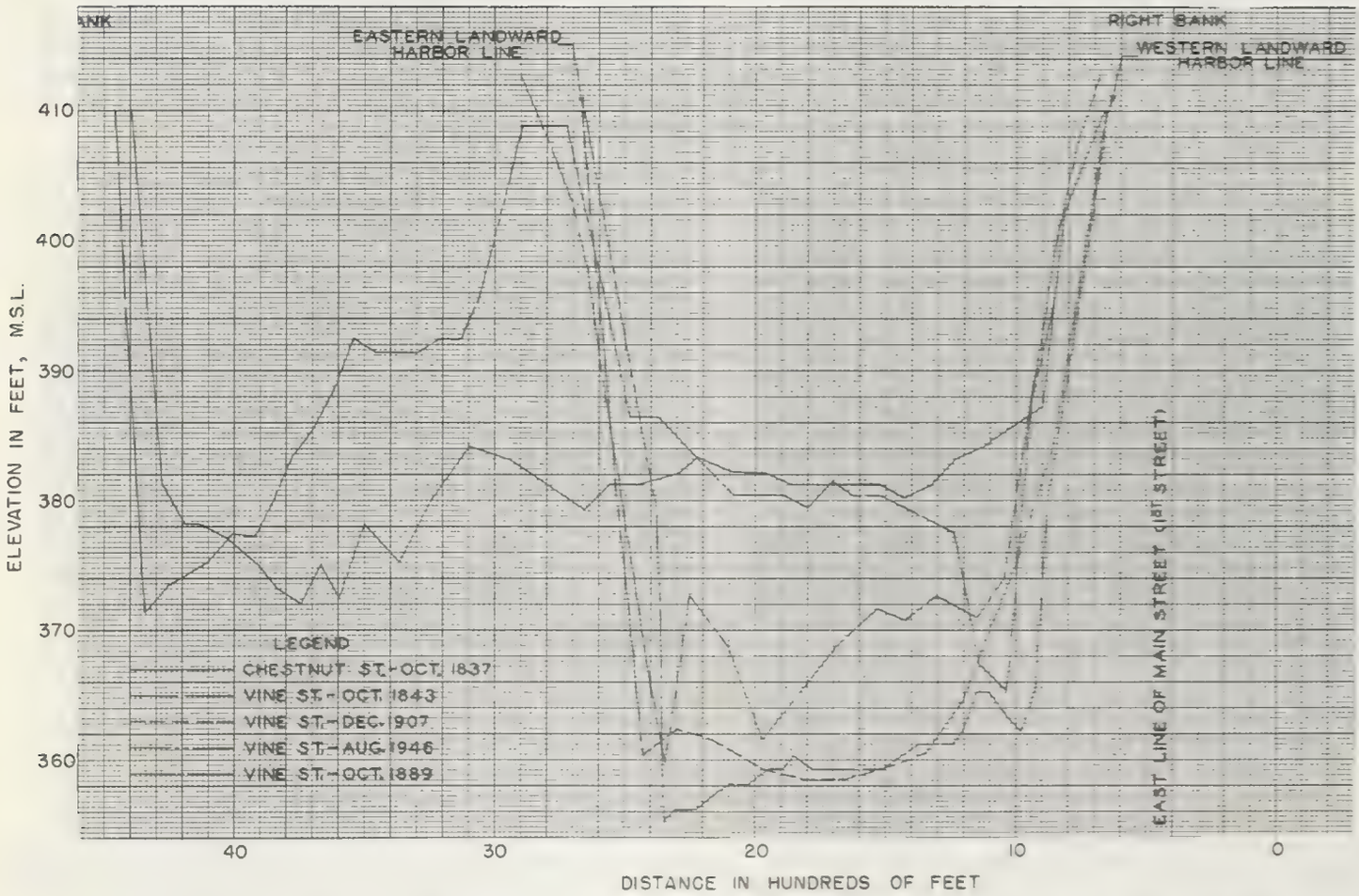


FIGURE 2. — Cross sections of Mississippi, downstream of Eads Bridge, 1837, 1843, 1889, 1907, and 1946.

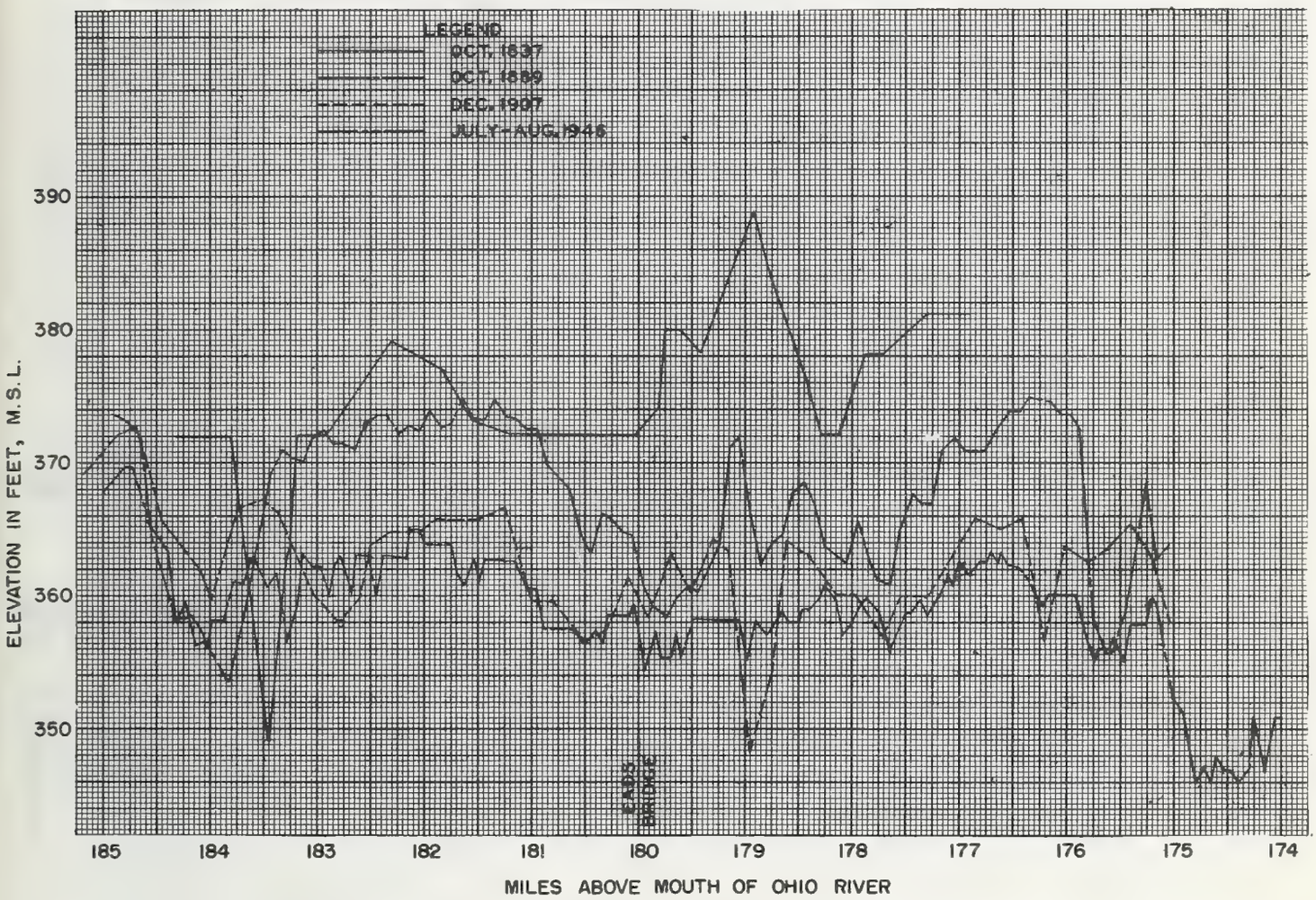


FIGURE 3. — Bed profiles of Missouri channel, 1837, 1889, 1907, and 1946.

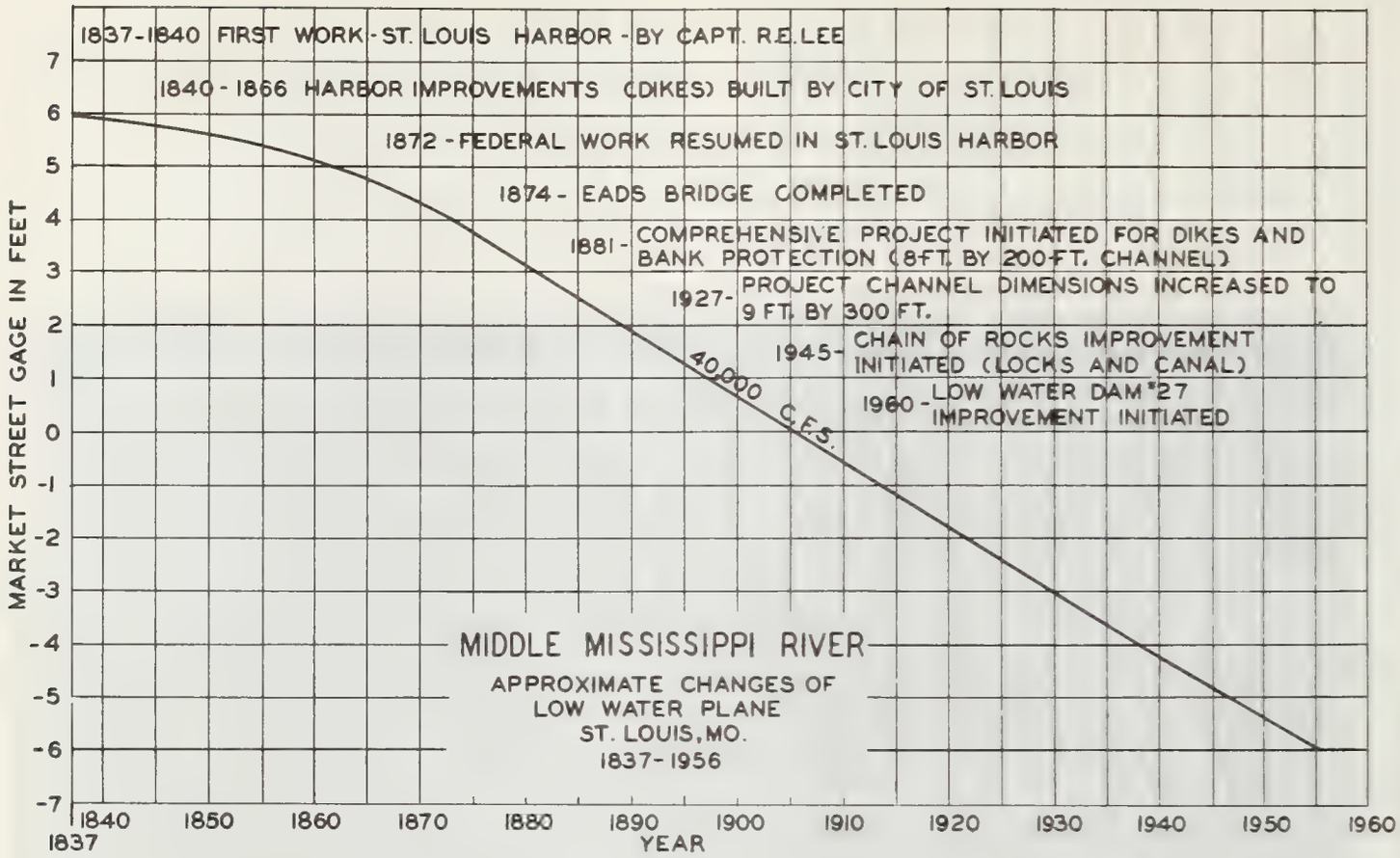


FIGURE 4. — Stage-time graph for 40,000 c.f.s., Mississippi River at St. Louis, 1837-1955.

the mouth of the Missouri. In the winter, generally, the upper Mississippi flow contribution at St. Louis is greater than the Missouri; consequently, in January 1952, this deposition moved through the St. Louis Harbor. Figure 5 shows comparative cross sections at mile 180.2, prior to and when the dome was moving downstream. The height of the dome was about 20 feet. A channel profile showing extent of and comparisons with previous profile is shown on figure 6.

In conclusion, it will be interesting to find out what affect the main stem Missouri basin reservoirs will have on the rate of long term change in the degradation of the channel at St. Louis, Mo.

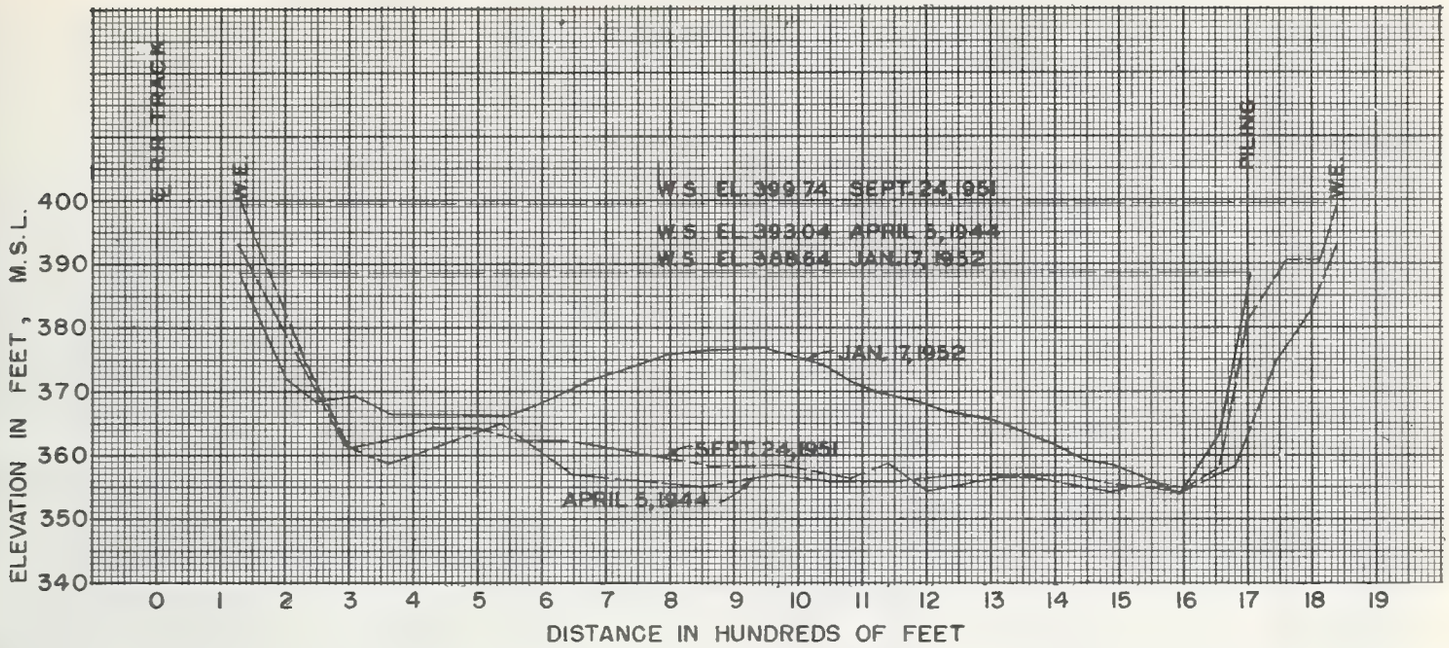


FIGURE 5. — Cross sections of Mississippi River at mile 180.2, prior to and after 1951 flood.

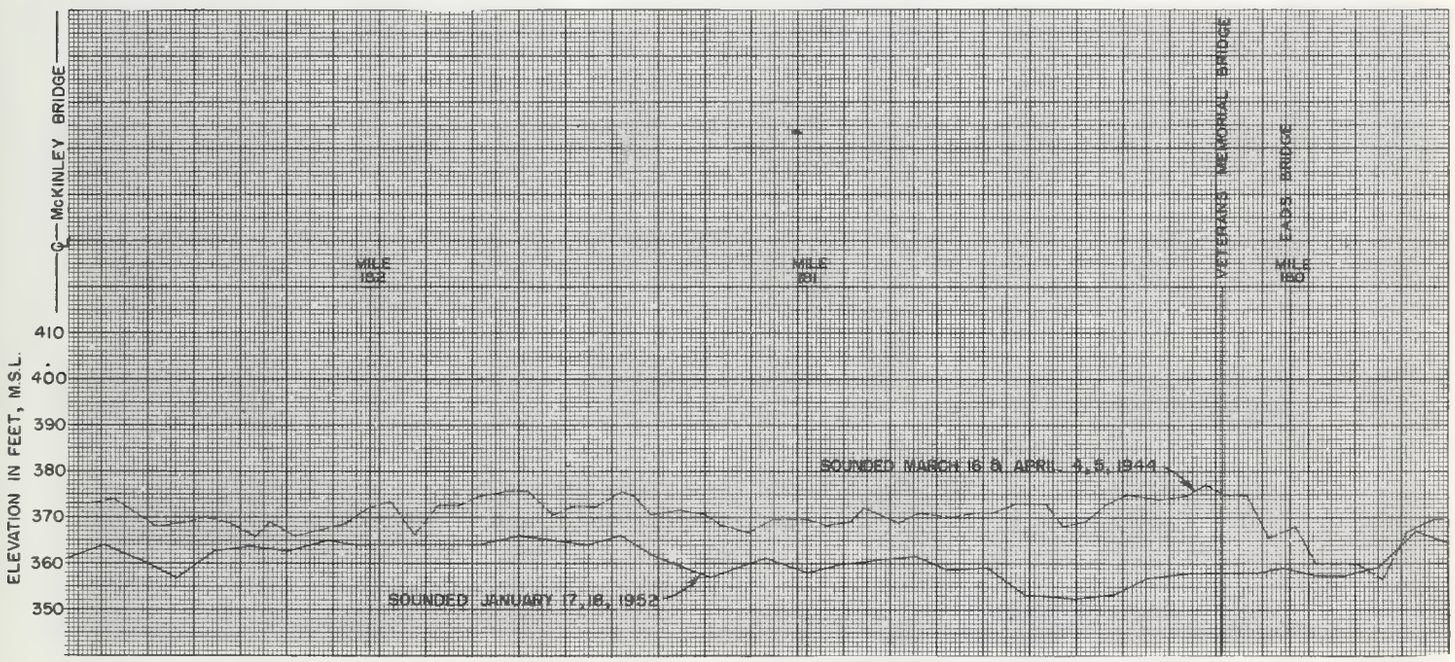


FIGURE 6. — Longitudinal channel profile, Mississippi River, 1944 and 1952.

TABLE 1.—*Discharge measurements, St. Louis, Mo.*

Date	Stage	Discharge[1]	Measurement method
	Feet	*1000 c.f.s.*	
14 Dec. 1880	5.4	51	Meter.
27 Jan. 1881	8.0	45	Meter (ice).
5 Feb. 1900	0.	35	Rod float.
7 Jan. 1901	.6	40	Meter.
16 Jan. 1902	1.3	44	Rod float.
22 Dec. 1903	2.3	51	Do.
21 Dec. 1904	.4	43	Do.
22 Dec. 1904	.2	39	Do.
3 Jan. 1905	1.5	54	Do.
16 Dec. 1910	−1.2	33	Do.
17 Dec. 1910	−1.5	34	Do.
31 Dec. 1910	.2	43	Do.
16 Jan. 1913	− .4	39	Do.
15 Jan. 1914	− .1	44	Do.
8 Jan. 1923	0.	48	Floats.
26 Dec. 1929	− .8	51	Do.
31 Dec. 1930	−1.3	41	Do.
3 Jan. 1931	−1.5	39	Do.
4 Jan. 1931	−1.0	42	Meter.
21 Oct. 1932	− .1	55	Floats.
1 Jan. 1934	−3.21	42	Meter.
31 Aug. 1934	−3.0	40	Do.
31 Aug. 1936	−4.0	38	Do.
16 Dec. 1937	−4.7	38	Do.
18 Dec. 1937	−3.5	40	Do.
2 Oct. 1939	−3.5	40	Do.
5 Feb. 1940	−3.3	34	Do.
13 Feb. 1940	−4.7	38	Do.
4 Feb. 1948	−2.5	50	Do.
28 Jan. 1954	−3.9	46	Do.
27 Jan. 1956	−4.4	45	Do.
8 Jan. 1959	−3.9	48	Do.
31 Jan. 1961	−3.9	49	Do.

[1]Discharge per foot about 5,000 to 6,000 c.f.s. at extreme low stages.

EFFECT OF A RIVER CONSTRICTION

[Paper No. 49]

By SAM SHULITS, *Professor of Civil Engineering, The Pennsylvania State University*

Abstract

The Susquehanna River at Middletown, Pa., is constricted considerably by a long embankment. The resulting downstream deposition has caused recorded progressive changes in the river course. Almost a mile of riverbank has already suffered substantial accretion that is expected to extend further downstream for miles with subsequent changes in the course of the river.

The Constriction and the Prior Condition of the Susquehanna River at Middletown

The Susquehanna River is constricted at Middletown, Pa., by a runway (levee) completed in 1957 (fig. 1). The total width of the river at Sassafras Island, originally 4,610 feet, is now 4,050 feet, which represents a narrowing of about one-eighth. The width of the channel between the left, or north, bank and the island has been reduced from 2,260 to 1,700 feet, corresponding approximately to a 25 percent reduction in width of that channel. The conditions in the slack water region just downstream from the constriction are the topic of this investigation. The study area is shown in figure 1.

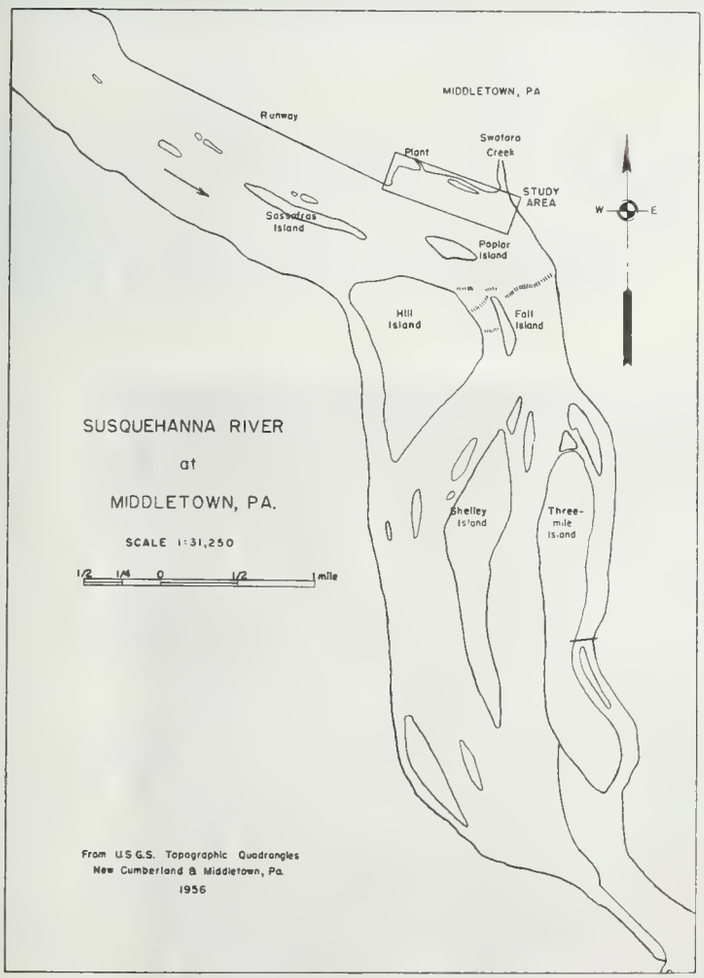


FIGURE 1. — Plan of Susquehanna River at Middletown, Pa., 1956.

The first aerial photograph of record is for the year 1937 (fig. 2). Noted should be the solitary island at the left bank between the intake and discharge channels of the electric generating station, and Swatara Creek (at the right of the photograph). This island will be named either "The Battery" or "Original Island" hereinafter. Subsequent aerial photographs taken in 1947, 1950, 1955, and 1956 show no discernible change in the river. Just upstream from the mouth of Swatara Creek in figure 3, there is a bar formation that should be borne in mind for subsequent discussion.

The incomplete constriction (white) appears first in figure 4, which was taken at a rather low flow (3,160 c.f.s.) on September 5, 1957. Here can be detected the first change in the slack water area; namely, a shoal about 200 or more feet riverward from the upstream end of the original island. The island and the bar at the mouth of Swatara Creek appear connected as part of the same river-developed formation also visible on figure 3, which was taken in 1950 at a flow of 6,780 c.f.s. This formation will play a role in the rationale to be developed later about the probable future configuration of the Susquehanna River.

Conditions Subsequent to the Constriction

The rapidity of the deposition in the front of the plant and just downstream from the levee after its completion is definitely demonstrated by figure 5, photographed in 1959, at a discharge of 3,950 c.f.s. As this is not an accurate aerial survey photograph, the magnitude of the accretion may be surmised from the 200-foot width of the runway at the lower left. The shoal mentioned above as barely appearing in figure 4 is now new land attached to the head of The Battery. The discharge from the plant, now trapped between the accreted land, the island, and the original shoreline, flows toward Swatara Creek. Formerly, this discharge flowed away from the shore out into the Susquehanna River. In figure 5 can be seen the bar extending

FIGURE 2. — Condition of Susquehanna River, November 23, 1937; discharge, 32,800 c.f.s.

from the toe of the embankment. Somewhere between this bar and the deposition will be the final or fully developed future left bank line of the river.

The photographic record of figure 5 is substantiated by surveys determining the riverbed elevation in the deposition area along transverse ranges (fig. 6). The first surveys in 1957 were not along these ranges, but by 1961 the ranges were established as a basis for future measurements. The ranges T–0 to T–26 became necessary when it appeared certain that the left or north bank was changing at least as far as Swatara Creek, a distance of about four-fifths

FIGURE 3. — Susquehanna River, August 13, 1950; discharge, 6,780 c.f.s.

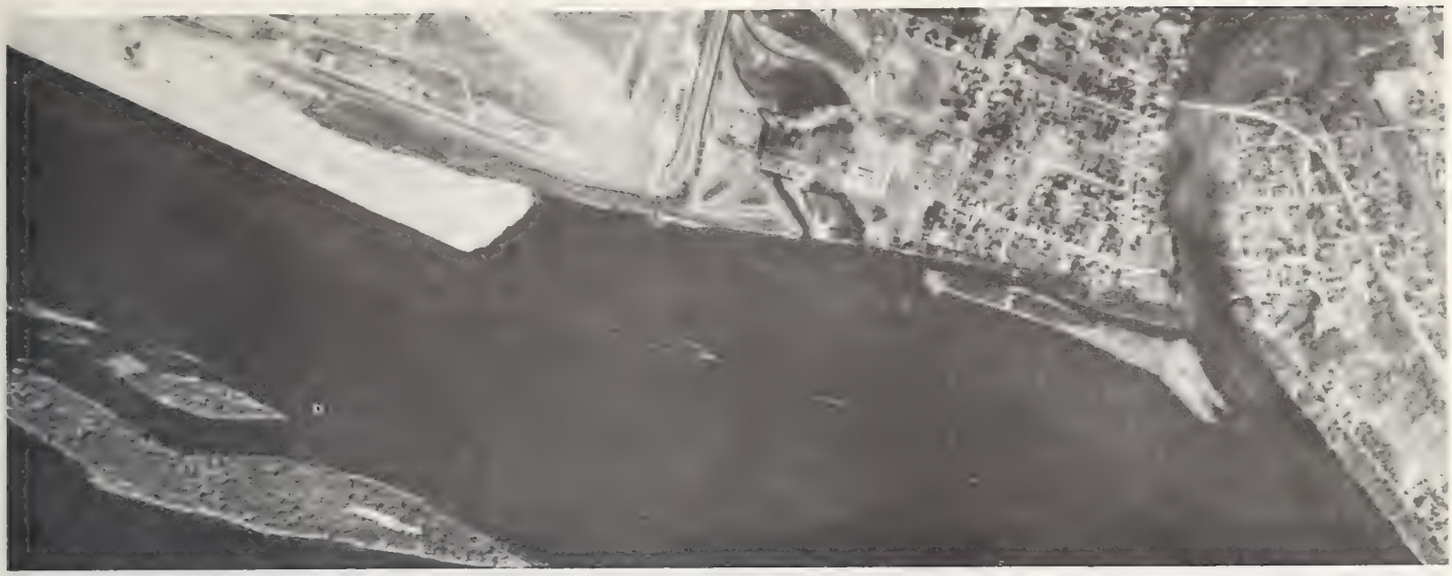


FIGURE 4. — Susquehanna River, September 5, 1957; discharge, 3,160 c.f.s.

of a mile. The T-ranges were used for the first time in August 1962.

The changes in bed elevation are exemplified by transverse profiles G and I (fig. 7). The earliest bed elevations of record are for April 1923. Systematic measurements of the deposition started in 1957. It is possible to reason from profiles G and I that a trough exists between 600 and 800 feet from the original shore and that this trough might not fill in. As there are only about four annual measurements as evidence of this trough, its permanence is not yet assured.

As the most data are available for ranges A to I, the area enclosed by these served as a basis for computation of the magnitude of the sediment accumulation. The mean total height of the deposit over this area is 0.15 foot for the period 1923-57, or an average annual deposit of only 0.004 foot. For the period 1957-62 the average accumulation is a total of 2.52 feet, or a mean annual deposit of 0.51 foot. The annual rate of deposit after the appearance of the constricting levee is about 130 times the mean rate for the 34-year period preceding the constriction.

The rapid occupation of the riverbed in front of the power plant by the accumulating sediment is represented graphically in figure 8, which shows the riverward advance of the shore line. Between ranges A and D the 1962 shore line is 550 feet riverward from the original shore line, and on range K it is 500 feet from the original shore line. This is a yearly advance of approximately 100 feet.

The measurements just described are not only a record of the effects of the constriction, but might serve two other purposes. First, although a prediction of the probable future course of the Susquehanna River will be ventured later, obviously only a model study could yield a truly reliable ultimate picture and for this purpose the measurements are indispensable. Second, it should be possible to relate the amount of sediment deposited to that transported in suspension by the Susquehanna River. It is reasonable to postulate that the sediment is deposited during the receding spring or snowmelt floods.

Probable Future of the River at Middletown

Before venturing to predict the future configuration of the river, particularly of its left bank, one should compare figure 2 portraying the preconstriction river and figure 9 depicting the river in June 1962. The shape of the deposition between the levee and the original island resembles closely the kind occurring downstream from river contraction works, such as spur dikes erected to induce deposition to protect or build up a riverbank. The shrubs on the bar at Swatara Creek are making it into a permanent island as is clearly evident on figure 10. That this bar could be suspected of acting as a spur dike is obvious in figure 11, where there are two smaller bars downstream from it. The Middletown Rapids stretch across the bottom of the photograph from the left bank to Hill Island (refer to fig. 1).

The foregoing conditions are made clearer by figures 12 and 13. The runaway width (200 feet) affords a sense of scale, but again it must be noted that these photographs are not accurate aerial survey photographs. The condenser

discharge water from the power plant flows between the original shore and the island, toward the bar at Swatara Creek.

With the aid of figure 14, which area extends eastward from the constriction to Swatara Creek and then southward to the Middletown Rapids, one may venture a prognosis of the long-range effect of the constriction on the pattern the river will follow. The shoreline in front of the plant will advance riverward at a decreasing rate, but probably not as far as the bar (fig. 5) extending downstream from the levee toe. The trough between the advancing shoreline and this bar could remain open. The riverward edge of the deposition in front of the plant might be a line from the toe of the embankment to the downstream tip of The Battery (original island).

Just as the bar at the toe of the levee showed shrub growths in the summer low-water of 1962, it is likely that the two long bars to the south in figure 14 will eventually trap some sediment when a lengthy low-water period happens and shrubs will develop on them. It is not too farfetched to conceive a chain of almost connected islands from The Battery to the bar formation at Swatara Creek to the two islands at the south. The discharge water will join Swatara Creek and flow southeast between the left or east bank and the suggested chain of islands.

FIGURE 5. — Deposition at Middletown, August 3, 1959; discharge, 3,950 c.f.s.

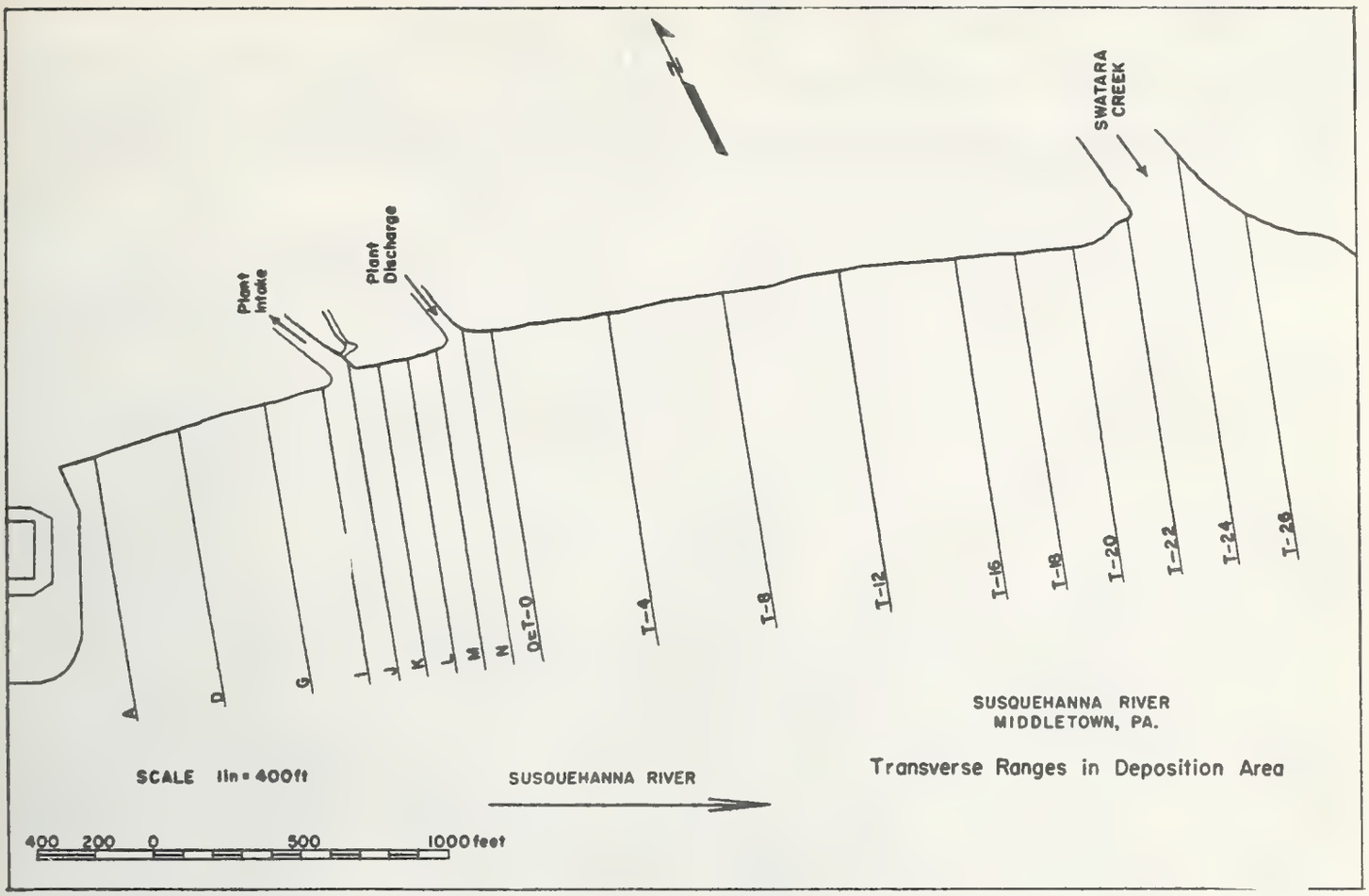


FIGURE 6. — Susquehanna River, Middletown; transverse ranges in deposition area.

The most conjectural part of this rationale is what the discharge water will do to the Swatara silt load. It is imaginable that the bar at Swatara Creek will grow northward to join the original shore, so that the discharge water would flow into the Susquehanna River at the downstream (east) tip of The Battery. Then Swatara Creek would have its own channel between the chain of islands and the east bank.

The prognosis of island formation extends only to the Middletown Rapids where the flow is accelerated and below which no investigations have been undertaken. Nevertheless, that the effect of the deposition at Middletown is reaching surprisingly far downstream is substantiated by the local observation that the main river current during the last 2 or 3 years seems to be between Hill and Shelley Islands (see fig. 1), whereas it was formerly between Shelley and Three-Mile Islands; that furthermore and consonantly the northwest side of Shelley Island is eroding. The reach between Hill and Shelley Islands is about 2½ miles around the bend from the downstream end of the constricting levee. As would be expected, the constriction itself is causing erosion of the bank of Sassafras Island facing it.

The separation phenomenon associated with the outer or concave side of a bend is obviously also an important factor in the deposition process. The river bend at Middletown has a central angle of about 130° (see fig. 1). The entire reach of the left or concave bank from the plant to the rapids upstream from Fall Island is in the separation zone below the runway where an intricate and connected system of slowly rotating eddies or rollers will provide a region for sediment deposition. That the constriction has enlarged the width of the separation zone is clear.

River Data

As an appendix some river data will be given. The slope of rock bottom is about 0.05 percent, or 0.05 feet in 100 feet. At a Susquehanna River discharge of 5,020 c.f.s., the water surface slope on September 26, 1961, was approximately the same, though some slopes of 0.03 feet in 100 feet were observed. The water surface slopes were measured out in the river just beyond the bar at the toe of the levee.

For the period 1957–62, the Susquehanna low water discharge in the summer varies from about 2,500 to 9,000 c.f.s. The peak of the spring snowmelt floods varies from about 170,000 to 376,000.

The 42-year average discharge of Swatara Creek is 572 c.f.s. For the water year 1960–61, the mean is 625 c.f.s. with a maximum of 9,340 and a minimum of 84.

The plant usage of water ranges from a minimum of 38.4 to a maximum of 366 c.f.s. The maximum usage is an inappreciable 15 percent of the lowest Susquehanna River summer discharge of 2,500 c.f.s. A condenser discharge

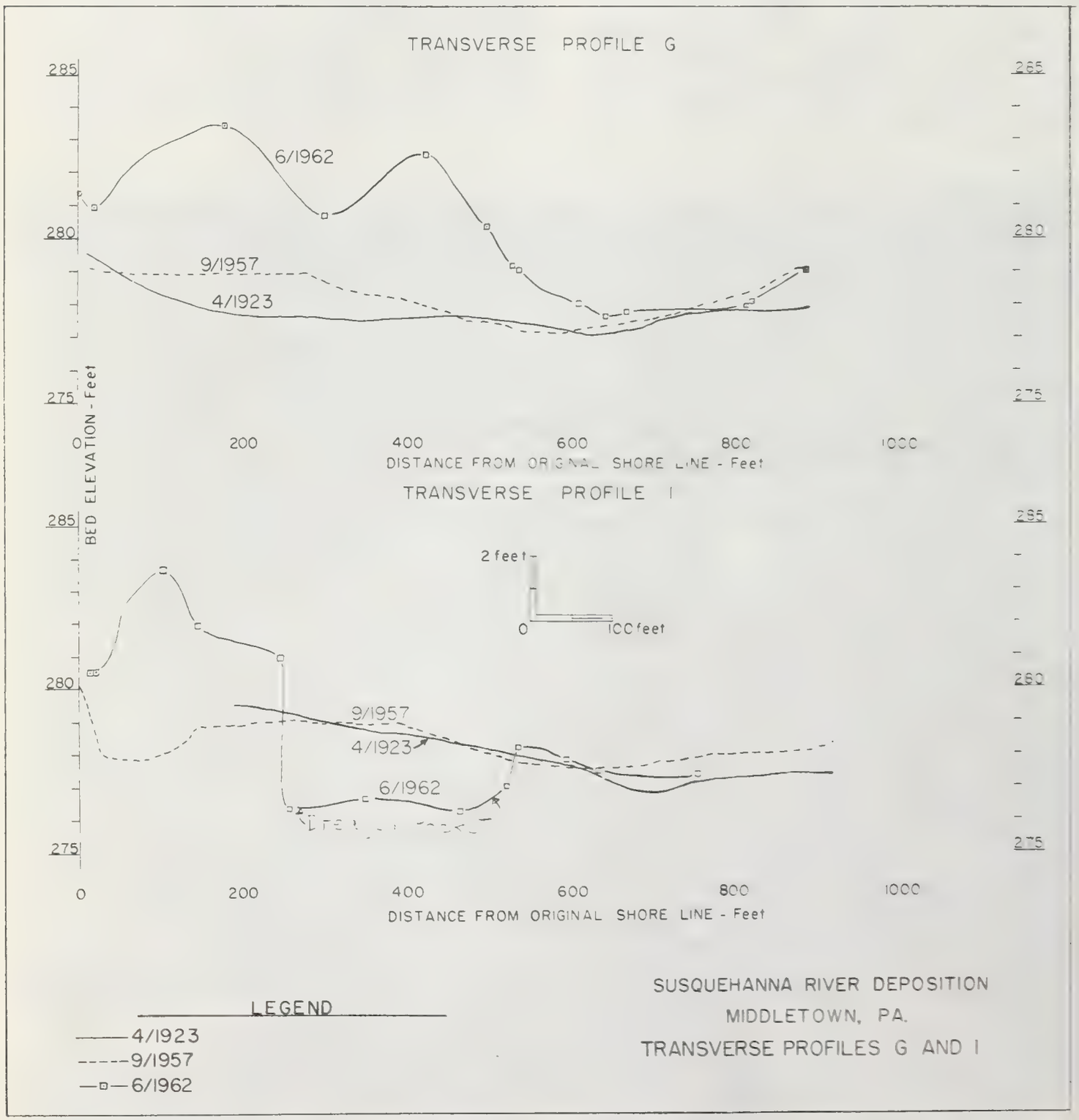


FIGURE 7. — Susquehanna River deposition at Middletown; transverse profiles G and I.

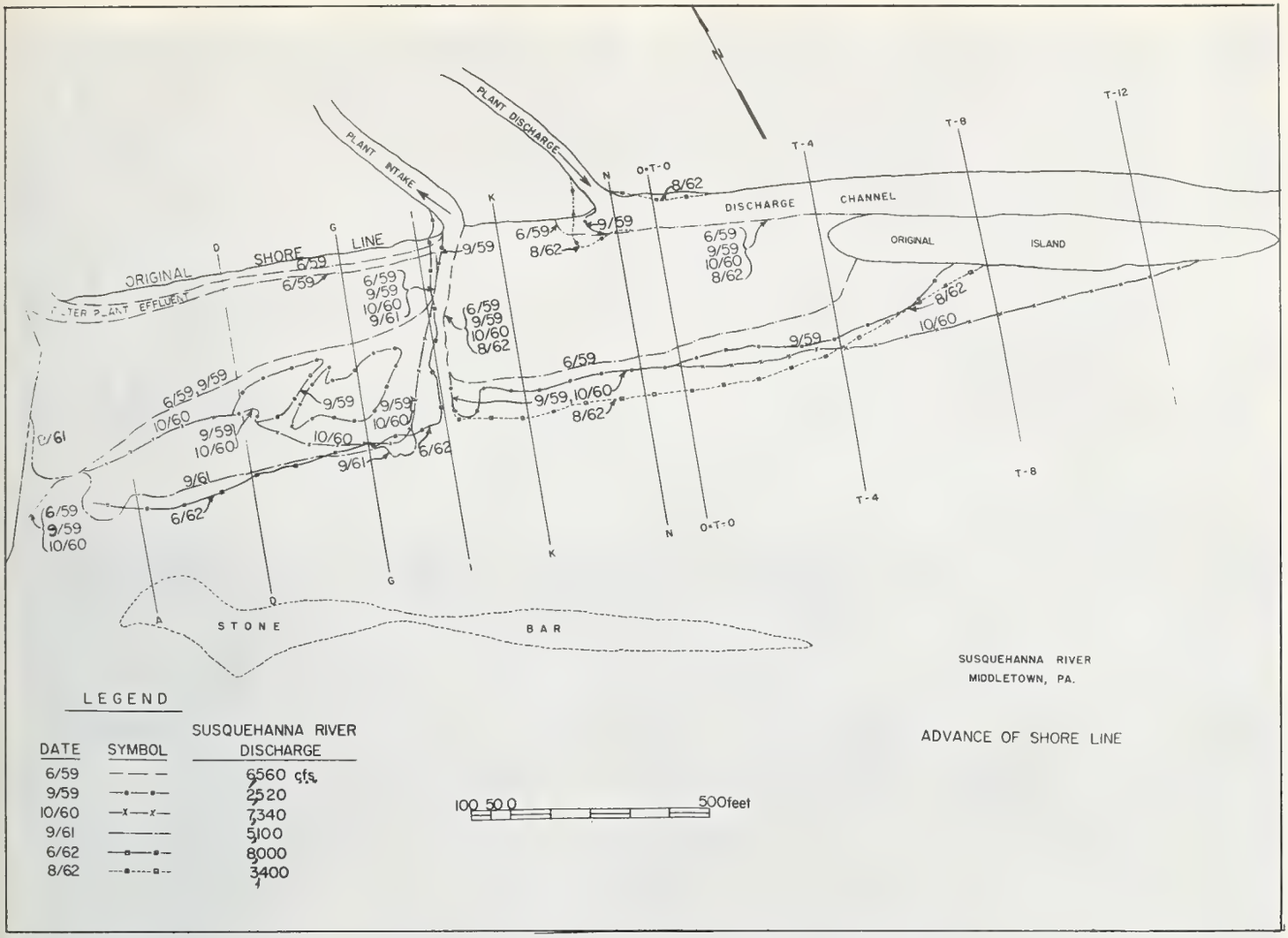


FIGURE 8. — Susquehanna River, Middletown; advance of shoreline.

FIGURE 9. — Susquehanna River, June 26, 1962; discharge, 5,860 c.f.s.

water flow of 366 could have considerable influence on the Swatara Creek.

The Susquehanna River suspended load measured at Harrisburg, Pa., ranges from approximately 7,000 tons per day at a discharge of 45,000 c.f.s. to 650,000 tons per day for 400,000 c.f.s. Swatara Creek at Middletown carries an approximate average of 660 tons per day.

Acknowledgments

J. J. Molloy, district engineer, Surface Water Branch, U.S. Geological Survey, Harrisburg, Pa., and his successor, R. E. Steacy, were very helpful in placing at my disposal discharge data for the Susquehanna River at Harrisburg.

J. R. George, geologist-in-charge, Quality of Water Branch, U.S. Geological Survey, Harrisburg, Pa., gave me important information on the suspended loads of the Susquehanna River and Swatara Creek and the geology of the area.

H. E. Bodden, superintendent of the Crawford Station of Metropolitan Edison Co. at Middletown, and D. G. Steck rendered me very substantial assistance at the deposition area by measurements, advice, and a steady flow of information; they were actually my watch on the river, and I would have accomplished much less without their continuous and enthusiastic cooperation.

Students of the Civil Engineering Department of The Pennsylvania State University assisted in the investigations; in particular, P. Pisaneschi, J. Smyth, and C. W. Lubold, Jr.

FIGURE 10. — Closeup of bar at Swatara Creek on Susquehanna River, June 26, 1962; discharge, 5,860 c.f.s.

FIGURE 11. — Susquehanna River, September 15, 1961; discharge, 5,640 c.f.s.

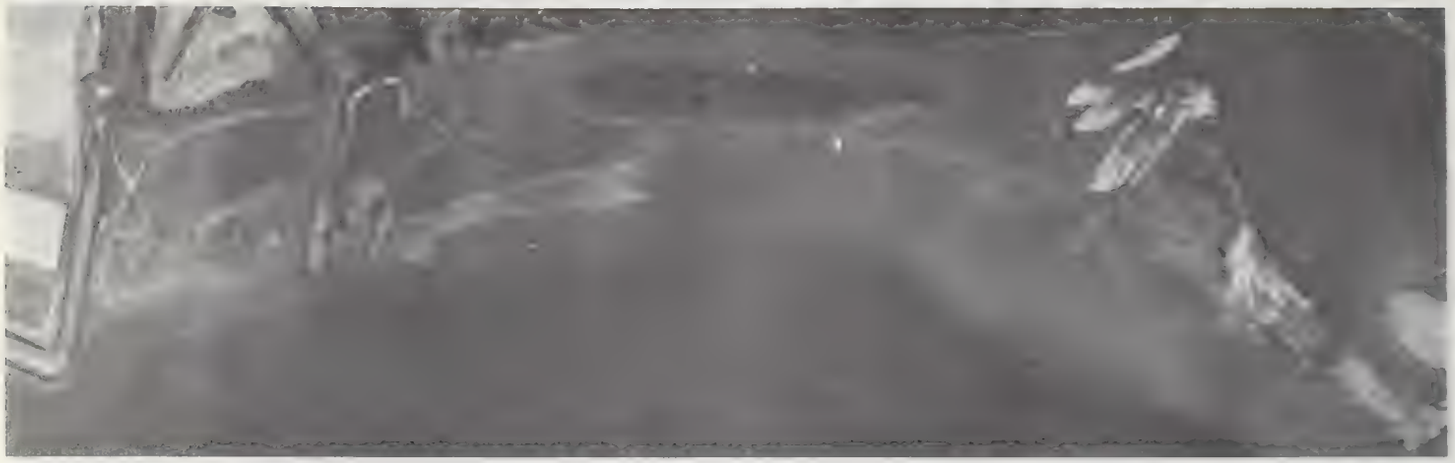


FIGURE 12. — Deposition at Middletown, September 23, 1959; discharge, 2,670 c.f.s.

FIGURE 13. — Bars at Swatara Creek, September 15, 1961; discharge, 5,640 c.f.s.

FIGURE 14. — Left bank of River susceptible to change, September 23, 1959; discharge, 2,670 c.f.s.

SOME ECONOMIC CONSIDERATIONS IN RIVER CONTROL WORK

[Paper No. 51]

By PAUL A. OLIVER, *chief, Division of Construction and River Control, Region 3, Bureau of Reclamation*

The name "Colorado River" and the term "muddy water" have long been linked in the minds of many Americans. When translated from the Spanish and applied to the description of rivers, the name "Colorado" means "reddish in color." Many will appreciate the humor of the expression "too thick to drink and too thin to plow" as often applied to the Colorado River. Prior to the closure of Hoover Dam these and many other terms justly described the river from its source to its mouth. However, since the closure of Hoover Dam in 1935 and the subsequent completion of Parker and Imperial Dams in 1938 and Davis Dam in 1953, these terms can no longer be justly applied to the river below Hoover Dam (fig. 1).

The changed regimen of the river resulting from the construction of this stairway of dams has been dramatic. For many years prior to the construction of Hoover Dam the river carried great quantities of sediment brought in from the drainage areas during the periods of floodflow. Some of this material was deposited in the river channel; some was carried to the delta located at the upper end of the Gulf of California. During the seasons after floodflows, part of the bed material was picked up and moved to deposits farther downstream. The net result was a gradual rise in the riverbed elevation in the lower reaches. Subsequent to the construction of Hoover Dam, the sediment load from the upper river has been trapped in Lake Mead and clear water released below the dam. This water, freed of the sediment load, rapidly scoured the deposit that was laid down during the earlier years. After the construction of the dams downstream, additional depositories for the scoured materials were formed and correspondingly more reaches of the river became subject to the erosive action of the clear water. The dramatic effect of the changed regimen upon the downstream sediment load can be expressed well in tabular form. Table 1, prepared from data given in Geological Survey papers, shows flows and sediment transport for two stations on the river. The Grand Canyon station, 267 miles above Hoover Dam, represents the near virgin condition, while the Yuma station, some 300 miles below Hoover Dam and located near the head of the Colorado River Delta, reflects the altered regimen.

To describe all economic aspects of river control for the entire reach of river between Hoover and the Delta would require more space than allowed here; however, some aspects are worthy of consideration. In this report, remarks will be confined to aspects of river control in the Needles area, the Blythe area, and in the Yuma area.

TABLE 1.—*Suspended sediment load of the Colorado River at Grand Canyon and at Yuma*

Water year	Grand Canyon		Yuma	
	Discharge, Acre-feet ×1,000	Load, Tons×1,000	Discharge, Acre-feet ×1,000	Load, Tons×1,000
1926	14,400	225,000	12,200	130,800
1927	17,300	398,000	17,100	242,500
1928	15,600	172,000	12,800	114,800
1929	19,400	480,000	17,500	289,000
1930	13,400	235,000	10,600	183,900
1931	6,700	68,800	4,800	52,100
1932	16,000	261,000	14,200	184,200
1933	10,000	178,000	8,000	92,200
1934	4,650	50,100	2,400	17,800
1935[1]	10,210	122,000	4,000	21,600
1936	12,300	158,000	3,500	14,000
1937	12,400	191,000	4,000	15,800
1938[2]	15,600	232,000	4,200	12,500
1939	9,620	86,300	6,600	13,000
1940	7,440	75,400	5,400	3,500
1941	16,900	270,000	11,700	14,100
1942	17,300	230,000	10,500	8,800
1943	11,400	58,800	7,300	3,000
1944	13,500	97,800	8,600	4,800
1945	11,900	83,600	6,500	4,400
1946	9,100	66,000	3,800	1,700
1947	13,700	136,000	4,200	2,000
1948[3]	13,900	144,000	6,300	6,700
1949	14,400	119,000	6,600	7,800
1950	11,100	59,800	3,500	3,500
1951	9,840	48,700	2,800	1,100
1952	18,200	148,000	9,200	10,500
1953	8,900	48,800	4,100	1,900
1954	6,200	40,700	3,200	1,300
1955	7,580	83,100	2,100	830
1956	8,700	76,100	880	260
1957	17,500	155,000	1,170	660
1958	14,500	132,000	2,950	3,900
1959	6,900	23,600	940	560
1960	9,580	37,500	700	137
1961	7,080	46,138	680	135

[1] Closure of Hoover Dam.
[2] Closure of Parker and Imperial Dams.
[3] Initial closure of Davis Dam.

River Problems, Needles Area

The problems in the Needles, Calif., area arising out of the changed regimen of the river are worthy of note, not only because they were the first to occur but because they were the most dramatic and costly to date. The Bureau of Reclamation studies indicate that from the time of closure of Parker Dam in 1938 until October 1944 a total of 84 million cubic yards of sediment eroded below Hoover Dam. Surveys showed that 67 percent of the total originated in the reach between Hoover and the site of Davis Dam and that the yearly contribution of

this reach averaged 500,000 cubic yards by 1944. Although a large part of the material eroded was deposited in Havasu Lake, some 48 percent of the total eroded up to December 1944 was deposited in the flood plain between Needles and Topock.

By 1941, however, the backwater arising from the filling of Havasu Lake had formed a pool over the flat bottom lands upstream of Topock. With the reduced velocities, the rate of deposition increased and, by the progressive creation and exposure of sandbars and growth of phreatophytes, the deposition was accelerated until by 1944 an almost impenetrable swampy jungle existed in the 12-mile reach between Needles and Topock.

As the backed-up waters reached the vicinity of Needles, lands and homes in the riverfront section of the town within the levee were inundated first by seepage and finally by backwater encroachment. Early in 1944 it became evident that the levee in front of Needles would soon be overtopped. Had this happened, inundations would have reached a stage endangering the municipal water supply, the California Pacific Utility Light and Gas Plant, and the Santa Fe Railroad Division Point facilities including the terminal, shops, roundhouse, powerplant, iceplant, and some 12 miles of double track main line.

To cope with this emergency, the Congress appropriated funds for raising the levee and for the purchase of land and homes that had been inundated in the city of Needles. Under the provisions of the Interior Department Appropriations Act of 1945, a total of $340,000 was made available, of which not to exceed $100,000 was to be expended for the purchase of land subject to seepage or overflow and for improvements thereon.

The history of changes in water surface elevation at Needles and at Topock, some 12 miles below Needles, is shown on the hydrograph, figure 2. The hydrograph shows that the rise that had been taking place at Topock prior to the construction of Hoover Dam continued at a gradual rate until Havasu Lake was formed. Within a short time thereafter the elevation of the water surface rose an additional 6 to 10 feet to about elevation 448.0 feet. On the other hand, the rise at Needles was halted almost completely after the closure of Hoover Dam and the level remained constant until 1941, when a rapid rate of rise began. This rapid rate continued at an average of 1.2 feet per year until the end of 1944 and elevation of 475.5 feet was reached. The level then proceeded to drop slowly until June 1951, when it was about 2.7 feet lower than at the end of 1944.

In 1942, an effort was made to scour the bars upstream from Topock by lowering the level of Havasu Lake. It was found, however, that although obstructions in the upper reservoir were removed during the lowering of the lake, little or no effect was noted on the gages above Topock and in the Needles area. The density of the growth in the swamp area had already reached such proportions that it was considered uneconomical to hold Havasu Lake at a low elevation long enough to reestablish a natural channel in the river. This decision was also reinforced by the fact that the gages at Needles responded directly to any change in release out of Hoover, and consequently it could be expected that additional sediment would act to reduce the effect of lowering Havasu Lake.

In an engineering study, completed in 1944, it was determined that the most practicable solution of the problem at Needles would be to dredge a channel and to open the way for the river to flow from Needles to Topock at a slope designed to provide for transport of all sediment. The completion of the dredged channel in 1951 between Needles and Topock and the subsequent adjustment of the river to this new route has resulted in the lowering of the water surface at Needles about 5 feet. In the years 1951–60, the dredge worked upstream and completed the channelization from Needles to a point some 8 miles downstream from Davis Dam. Concurrently with the dredging of the channel a program of bank line revetment and stabilization was accomplished. As a result of this dual program the erosion of material along the new channel is largely confined to that in the bed of the river. The dredging operation was conducted so as to take advantage of the larger deposits of gravel in the cut to establish sills for grade control, and the bottom is gradually becoming armored in other reaches with heavier material as the finer sediments are removed by the river.

Studies made in support of an extension of channelization upstream from Needles indicated that a total of 112,261,000 cubic yards of sediment would be eroded from this reach of river if left in its natural state. Continued deposition of this material in the upper 12 miles of Havasu Lake would have caused a restoration of high-water conditions at Needles. As a result of channelization, the total suspended load, in tons per day at 15,000 c.f.s., has decreased at the Needles station on an average from 14,000 tons per day in 1956 to 5,000 in 1962.

In order to reduce the possibility of a later buildup of sediment downstream at Topock and to reduce the amount of material carried into Havasu Lake, it was decided to create a settling basin immediately upstream from Topock in the newly dredged channel and to station a small

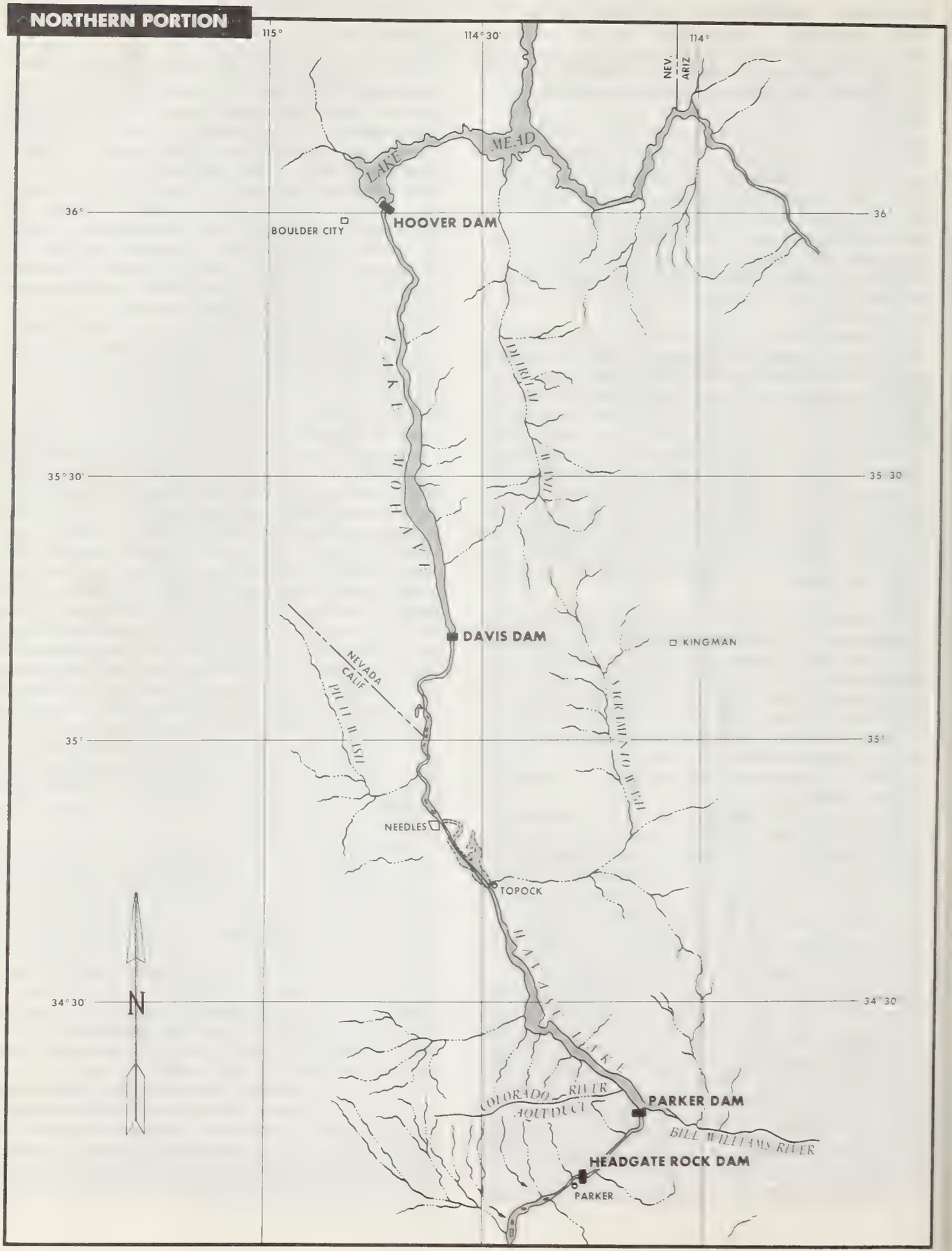
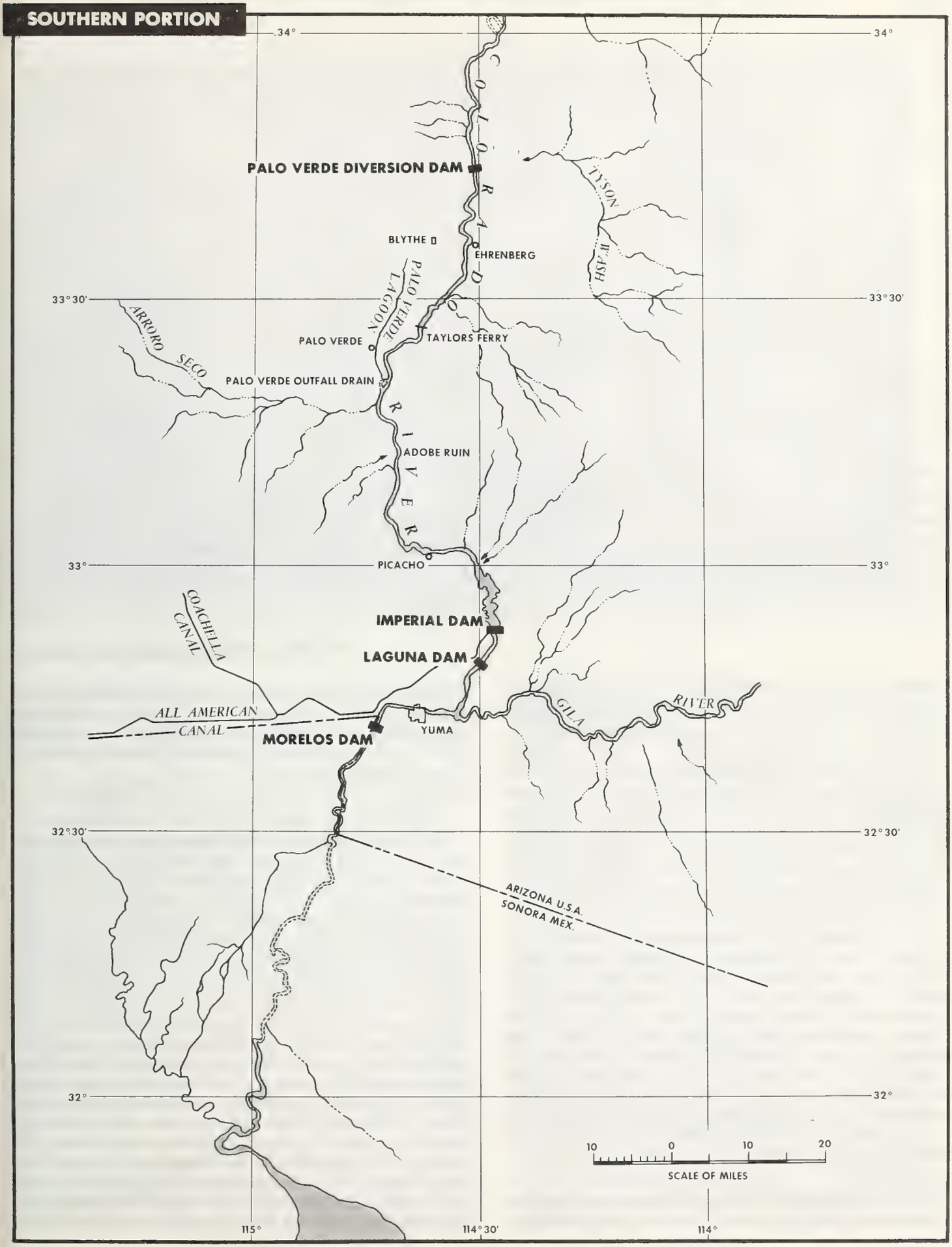


FIGURE 1.—Location map



of Lower Colorado River.

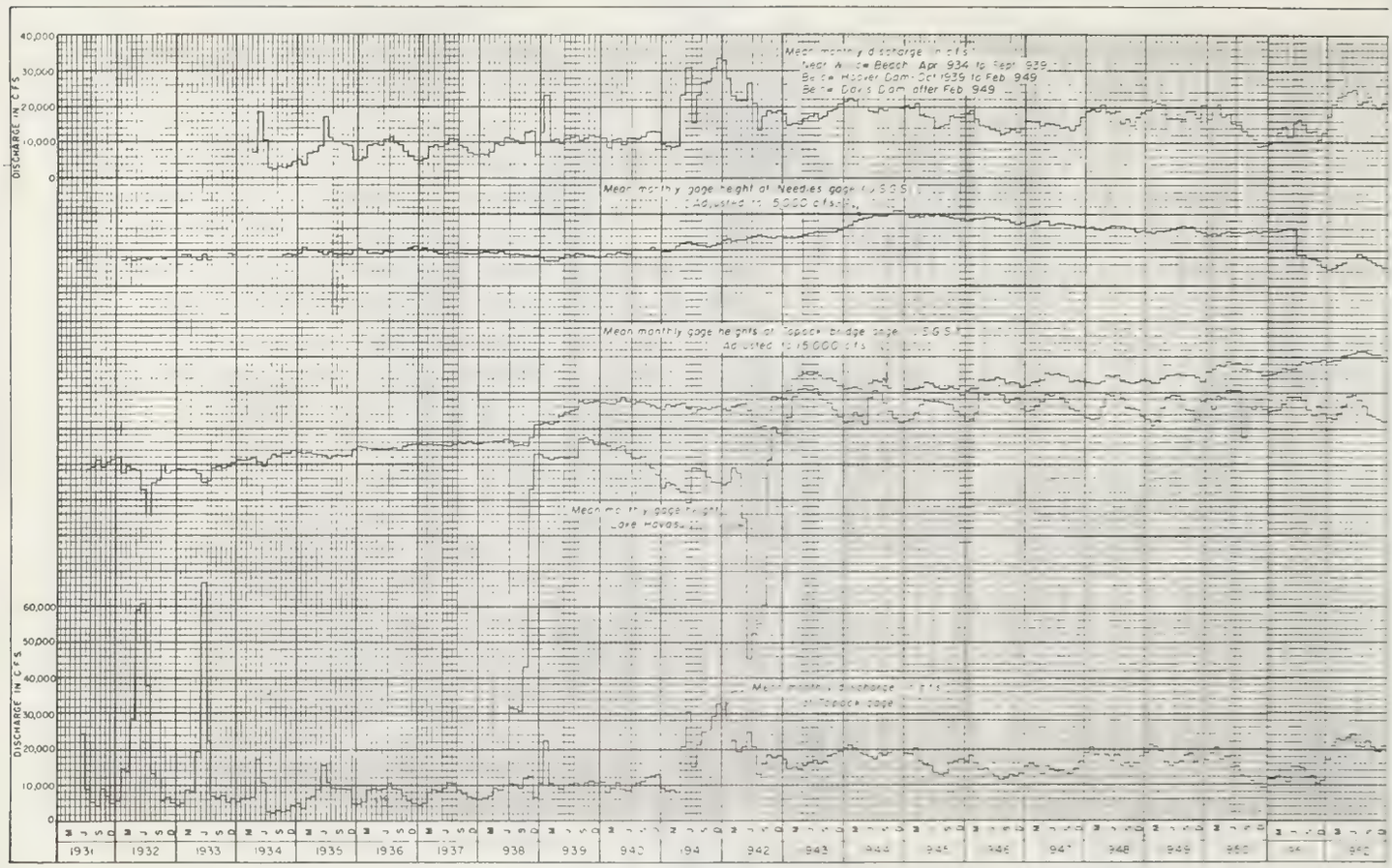


FIGURE 2. — Discharge and water surface

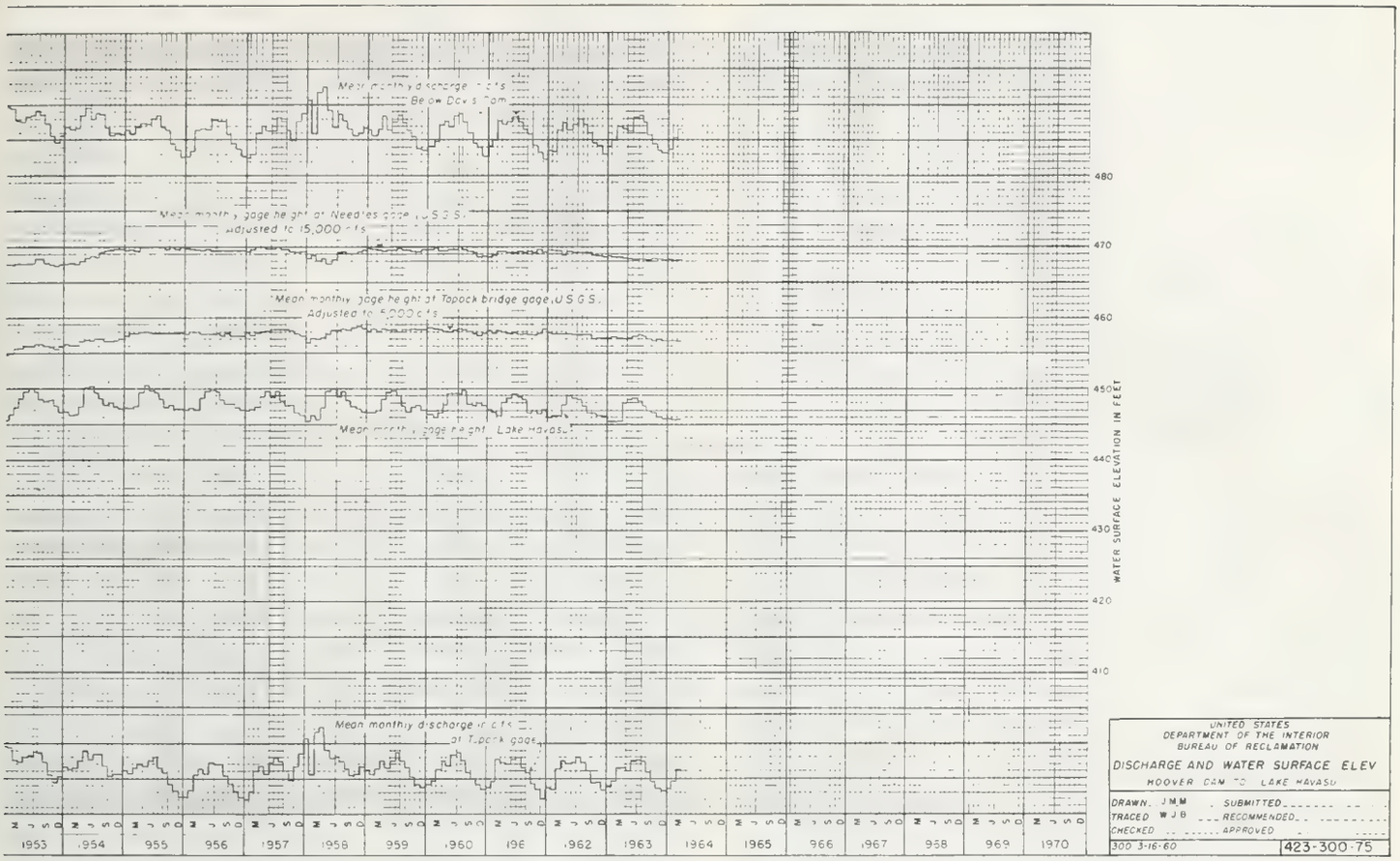
dredge at this point for removal of sediments arriving at that point. A 12-inch dredge was acquired in 1957, and since that time it has been working back and forth in this settling basin, removing the sediments arriving therein. Material removed is being deposited in the swamp area where it acts to reduce to some extent the open water area and tule growth. By 1965 reduction in sediment transport should allow the moving of this dredge to immediately below Topock, where a new program directed to channelization can be initiated.

It is expected that completion of the work in this region will produce a channel slope from Davis Dam to Havasu Lake that will be adequate to carry the relatively clear water flow anticipated at that time. Work to date in the stream between Davis Dam and Topock has resulted in the estimated annual salvage of some 65,000 acre-feet of water, and cost of the rechannelization has totaled $7,500,000. The estimated cost of channelization through Topock Gorge is $5,000,000, and the additional salvage in water should total another 60,000 acre-feet per year.

The studies mentioned previously predicted a cost–benefit ratio for work for the area between Davis Dam and Topock as summarized below. Recognition was given in these supporting studies of an inseparable relationship between work in this area and work to be done in the Topock Gorge.

Period:	*Benefit-cost ratio*
100-year	2.18 to 1
50-year	1.78 to 1

A revised program has since been developed for the Topock Gorge (below Topock), and a new overall cost–benefit ratio remains to be determined. In the original benefit study, no consideration was given to flood control and sediment reduction, as it was found that the benefits from other factors greatly exceeded the expected project costs, even though the relatively conservative values for irrigation use were applied to the salvaged water. It is expected that use of values attributable to municipal and industrial use will be considered in the later studies, and, if needed, the benefits from sediment reduction and flood control will be evaluated. Although flood benefits were not considered, an idea as to the threatened cost of damage to the Santa Fe Railroad should be mentioned. If the uncontrolled rise in river levels opposite Needles had been left to continue and thereby to force the relocation of the Santa



elevation at Needles and Topock gate.

Fe Railroad facilities, the cost of the relocation, estimated in 1947 by the chief engineer for the company, would have been somewhere between 10 million and 12 million dollars.

Not only has much of the agricultural area that was inundated by the high water been redeveloped, but there have been ancillary benefits arising from expansion of the recreational use of the riverfront lands. With the advent of clear 50° to 60° temperature water below Davis Dam, the fish and wildlife interests have introduced trout into the stream and a new type of settlement pressure has been the result. Where previously the lands were utilized only for farming, several full sections have been subdivided into smaller acreages for homesite and trailer park development.

The increased recreation pressure has resulted in the development of several bypassed inlets and lagoons along the new channel into small boat and trailer park marinas through State, county, city, and private enterprise. The mild winter climate that prevails has also had its influence. All these factors have acted to increase the rate of development and accordingly the value of lands along the river. Records in the assessor's office attest to the increase in prices paid for river frontage. Prices rose from approximately $10 to $20 per front foot in the mid-1950's to more than $100 per front foot by 1960.

River Problems, Blythe Area

The town of Blythe, Calif., and its principal source of economic support, the Palo Verde Irrigation District, lie about midway in the 150-mile reach between Parker and Imperial Dams. After the closure of Parker Dam in 1938, difficulty opposite in nature to that at Needles developed at the Palo Verde Irrigation District intake, 65 miles downstream from Parker Dam. Degradation of the riverbed lowered the water surface to such an extent that by 1944 gravity diversion into the District's canal was no longer possible. In 1944, the Congress approved funds for construction, operation, and maintenance of a temporary rock weir designed to restore operating levels at the intake. This weir, which was completed in 1945, initially required a total of 40,000 cubic yards of rock to raise the water surface to the required level for gravity diversions.

However, before the weir was replaced in 1957 with a permanent structure, the original total of rock placed had been increased by nearly 200,000 cubic yards as a result of need for re-

pairs and reinforcement. The permanent structure consisting of a rockfill dam, headworks, and a levee and drain system was constructed in accordance with the provisions of a repayment contract between the United States and the Palo Verde Irrigation District. Under this contract the United States agreed to construct the diversion dam, spillway, and intake works at a cost not to exceed $4,538,000 and to loan the District $500,000 for use in revising its distribution system to conform to the new diversion works. In addition to furnishing the necessary right-of-way and agreeing to operate and maintain the completed works, the District agreed to repay $1,175,000 in addition to the $500,000 as its share of the cost.

As a result of the combination of degradation and aggradation and the sill effect of the Palo Verde Diversion Dam, a degree of stability now exists in the riverbed for the 150-mile reach between Parker and Imperial Dams. The reduced slopes and the exposure of armoring beds of coarse gravel in the upper reaches have served to greatly reduce scour, while the decreased erosive force of sediment laden water has been the principal factor in reduction of scour in the downstream part.

Despite the quasi stability of the bed, an active erosion and sedimentation problem of considerable magnitude exists in this 150-mile reach. Degradation of the upper 90 to 100 miles has resulted in rather severe entrenchment, and, as the clear water is continually attacking the high banks, the sediment contribution is rather large. Water loss is also quite high in this 150 miles as a result of the excessive amount of braided, shallow, and excessively wide channel areas existing through the reach. Water losses are magnified in the downstream part where the river is contributing to high-water tables and excessive growth of phreatophytes.

Reconnaissance studies recently completed indicate that reduction in sediment transport and salvage of water in this reach can amply justify a channel improvement program, which it is estimated could cost $16,000,000. As a result of comprehensive studies, part of this program is already underway and further studies necessary to firm up the estimates for additional works are in progress. The complete program is expected to halve the 3 plus million cubic yards per year total of sediment now moving from this reach into the Imperial Dam reservoir and into the sediment control facilities of downstream irrigation districts. The confinement of the flow in a properly designed channel with stabilized banks and a gradient that will provide improved drainage of irrigated acreages should reduce the present average annual water loss by approximately 90,000 acre-feet.

Recreational and senior citizen groups are already taxing the marina and trailer park facilities in this area, and plans for the expansion of these facilities are being pressed by State, county, and private interests. It can be expected that upon the completion of the programed river control improvements, further expansions and attendant benefits to the local economy will result.

River Problems, Yuma Area

The Yuma area now marks the end of the line for the Colorado River. The last available water is diverted from Imperial Dam 17.5 miles upstream from Yuma for the irrigation of about 800,000 acres of land lying in the United States and from Morelos Dam 8.3 miles downriver from Yuma for the irrigation of about 400,000 acres in Mexico.

After closure of Imperial Dam in 1938, releases from Hoover Dam exceeded the downstream diversion requirements for several years and, as a result, the clearer water leaving Imperial Dam caused a degradation of the riverbed opposite the intake to the Alamo Canal, which supplied water to Mexico. By 1944 it became impossible to divert the required amounts, and service to Mexico was supplied by diversion from the All-American Canal to the Alamo Canal under special arrangements each year until 1950, when Morelos Dam was put into service by the Mexican Government in conformity with provisions of Article 12a of the February 1944 Treaty with the United States.

In the low-water years beginning 1953, a serious river regulation situation has developed. Referring again to the table on page 1, the average annual flow in the river at Yuma has dropped below a million acre-feet per year. This flow, plus returns to the river at points downriver, makes up to the guaranteed 1.5 million acre-feet that this country delivers to Mexico under the Treaty of 1944. Since 1953, in only 2 years have flows passed the Yuma gaging station in sufficient quantity to cause degradation of the channel. Stored water has been insufficient in the remaining years to provide regulatory flow sufficient to move downriver all sediment returned to the river below Imperial Dam and general aggradation of the riverbed resulted.

Since 1950 the Alamo Canal immediately below the inlet works at Morelos Dam has been maintained as a settling basin and cleaned periodically by dredging. By 1956, however, the spoil area had been built up so high that the 12-inch dredge maintaining the desilting basin

could no longer economically pump the silt to the designated area and the Mexicans have since deposited some 61,000 cubic yards of dredged material in a relatively short stretch of river below Morelos Dam.

Efforts to remove the channel constricting deposit hydraulically by means of large releases of stored water have met with only limited success and such efforts have been abandoned. Negotiations are now in progress to have the material removed by mechanical means and for the Mexicans to seek other spoil areas on land for future operations.

The limited releases from Imperial and the practically nonexistent flows below Morelos have resulted in an alarming decrease in carrying capacity of the floodway below Imperial Dam. An attempt this winter (1962–63) to improve the channel by release of some 270,000 acre-feet of stored water in regulatory flows has met with only nominal success, and a program of channel rehabilitation is planned by both the Bureau of Reclamation and the International Boundary and Water Commission.

This program will include the stationing of a 12-inch dredge, now under construction, in a swampy area below Imperial Dam where a desilting basin is planned. The silt released below Imperial will be trapped in the basin and deposited by the dredge in the adjacent swamp area. Clearing of the floodway will be initiated in the next fiscal year, and, in the following years, an overall program will be directed toward construction of a low-flow channel and a well designed floodway between Imperial and Morelos Dams. The International Boundary and Water Commission, United States Section, is negotiating with the Mexican Section to provide an extended international program of channel clearing and floodway maintenance in the 40-mile reach of river below Morelos Dam.

The daily scheduling of water to arrive at Imperial Dam for delivery to the 1,200,000 acres of land in the United States and Mexico is being subjected to considerable scrutiny in view of the recent history of low runoff, the impending closure of Glen Canyon Dam, and the need for filling Lake Powell. The fact that the final control for flows reaching Imperial is at Parker Dam 150 miles away along a badly deteriorated channel causes continuing concern to water-scheduling people. In 1961, the best year of operations, delivery of 7,000,000 acre-feet of water was made at Imperial Dam and the flow deviated only 5,000 acre-feet from the amount scheduled for arrival.

Cause for concern arises from the fact that even with this close limit on delivery, some 300,000 acre-feet of water over and above that ordered by Mexico under the Treaty was delivered at Morelos. As most of the yearly overdelivery generally consists of cutbacks in American orders and unforeseen changes occurring in the 3 days required for flow to reach Imperial Dam from Parker Dam, reregulation at Imperial is imperative. It is believed that this can be accomplished through construction of an offstream storage reservoir in a side canyon 1½ miles upstream from Imperial Dam. Feasibility studies show that a 15,000 acre-foot storage reservoir, having a 500 c.f.s. pumpback storage facility, will economically salvage a substantial part of the 300,000 acre-feet of overdelivery presently being wasted to the Gulf of California. The estimated cost of construction of this offstream reservoir and pumping plant is $7,000,000.

The economic expansion of the southwestern part of the United States is directly related to the availability of a dependable supply of water. Although the limit of supply of stored water in the Colorado River Basin is near, the fact is recognized that there is a vast possibility for economies in the use of this resource. This paper describes some potential sources of salvage; however, the Bureau of Reclamation estimate of the total potential salvage along the river and in dependent irrigated areas is in the realm of 1,000,000 acre-feet. This total could no doubt be augmented somewhat in practice if a sincere effort is directed, at all points from storage dams to the last irrigator on the lateral, toward economic use of water. The Bureau of Reclamation, as representative of the Secretary of the Interior in custodianship of the Colorado River, welcomes the responsibility of leading an all-out effort toward the salvage of water now being lost.

SEDIMENT DEPOSITS ON THE CONE AREA OF BIG SAND CREEK, AT VALLEY HILL, MISS.

[Paper No. 52]

By Fred Bayley, Jr., *U.S. Army Engineer District, Vicksburg, Miss.*

Synopsis

Hill tributaries of the Yazoo River system in Mississippi carry a tremendous sediment load. This tends to be deposited near the foot of the hills in large cone-shaped fills as the streams enter the flat alluvial valley. Big Sand Creek in north-central Mississippi is typical of these. It affords a rare instance where estimates of sedi-

ment rates can be based on actual field measurements of deposits. Sediment deposits on the cone area have been computed from a comparison of range profiles obtained in 1920 with those made in 1947 and 1957. This paper describes the watershed, the range profiles, and the amount of sediment deposits. Between 1920 and 1947, these deposits amounted to nearly 1,000,000 cu. yd., or 583 acre-ft. per year, equivalent to over 5 acre-ft. per square mile per year.

General

The line of hills along the eastern side of the Mississippi Delta rises abruptly about 150 ft. in elevation in a very short distance. The streams, heavily laden with sediment, enter the alluvial valley where the flat slopes drastically reduce the sediment-carrying capacity. Thus, a major part of the sediment load is dropped near the foot of the hills to form the cones peculiar to the area. These cones rise considerably above the surrounding land and fan out in a westerly direction. Sediment deposits on the cone area of Big Sand Creek, a hill tributary of the Yazoo River, are typical. The magnitude and rate of these deposits were computed from range profiles made in 1920 and those obtained 27 years later, in 1947, and 10 years thereafter, in 1957. This paper briefly describes the watershed, the range profiles, and the amount of sediment deposits. Figures 1 and 2 show the sand bottom and the caving banks of Big Sand Creek. Figure 3 shows typical sand deposits around cabins.

Watershed Conditions

Big Sand Creek lies in Carroll and Montgomery Counties, Miss., and flows in a westerly direction, entering the Yazoo River at Greenwood, Miss. Figure 4 shows the drainage basin. The hill area of the stream, which lies east of Valley Hill, is about 18 miles long and contains 110 square miles. Elevations in this area vary from 150 ft., mean sea level, near Valley Hill, to 520 ft., mean sea level, at the northeast divide. Soils in the upland part are within the loess resource area. The loess is quite deep in the western part and decreases in thickness to the east.

FIGURE 1. — Channel of Big Sand Creek 2.5 miles west of Carrollton.

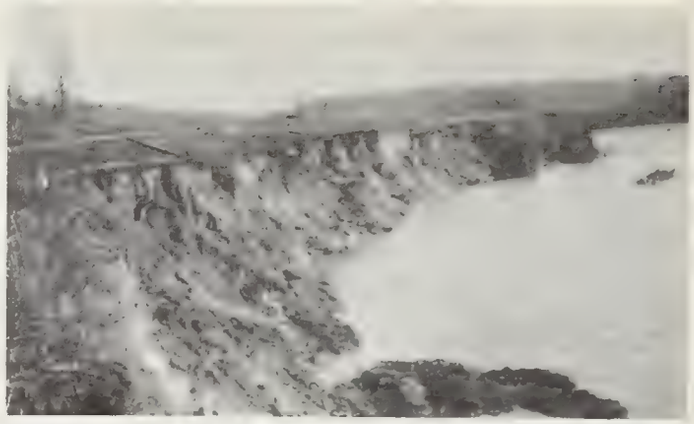


FIGURE 2. — Bank of Big Sand Creek near Valley Hill.

FIGURE 3. — Typical sand deposits.

The streams and large gullies have eroded through the loess into the sands and clays of the underlying Coastal Plain formation. The easy erodability of these soils is a large factor in the huge amount of sediment deposits in the delta area. Practically the entire upland watershed is affected by sheet and gully erosion, which is most severe in the area east of Carrollton.

The delta area extends west of the bluff line near Valley Hill and contains the alluvial cone on which the range profiles were made. The local drainage district constructed a channel and levee system in the delta part in 1922–23 in an attempt to carry flows from the hills to the Yazoo River. The carrying capacity of these levees rapidly deteriorated because of large sediment deposits in the floodway, until at the present time they are crevassed by flows exceeding about 5,000 c.f.s. The levees are con-

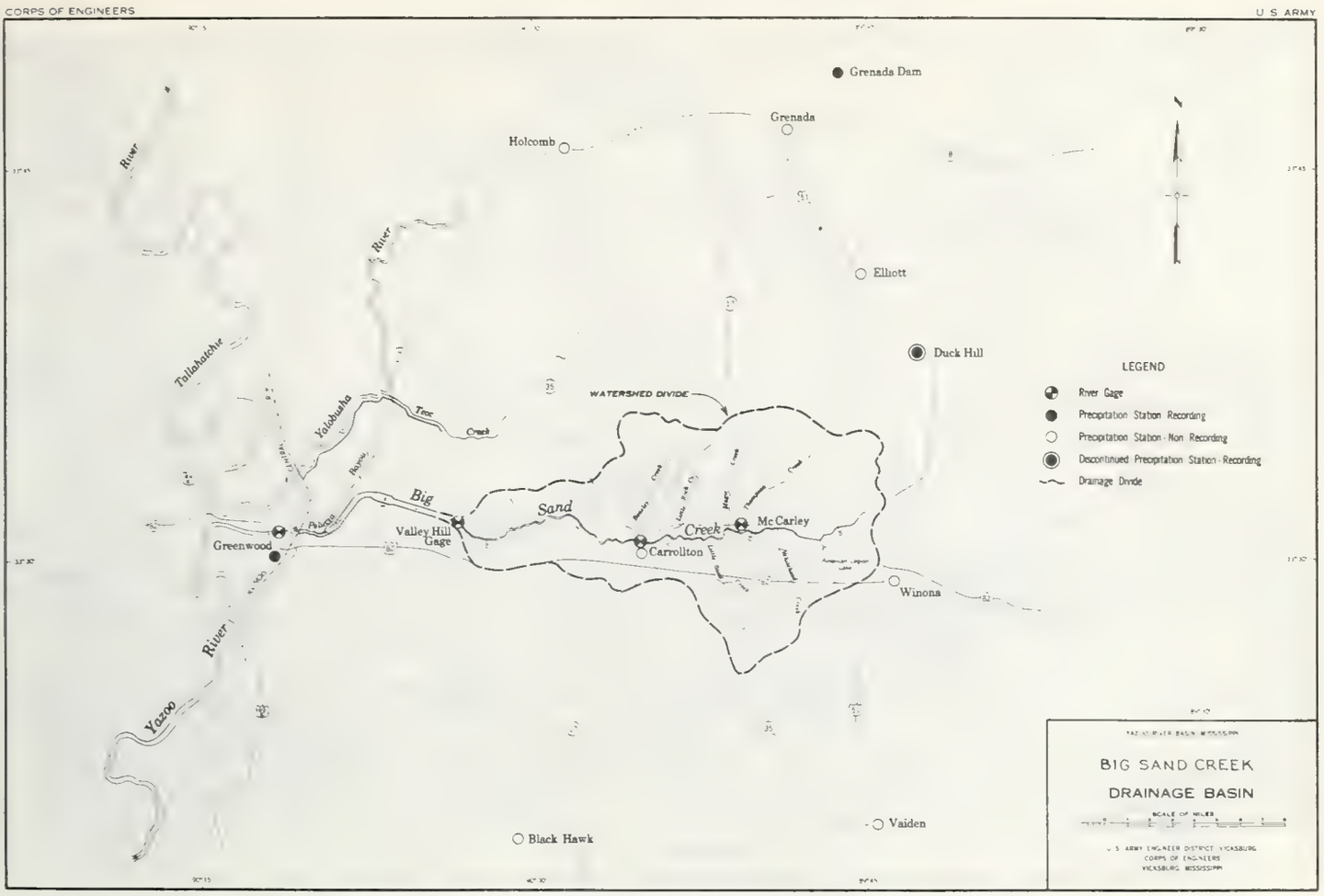


FIGURE 4. — Drainage basin of Big Sand Creek.

structed of sandy material and crevasses occur almost every year, with several in some years. The poor soil cover in the upland area is conducive to rapid runoff, with factors as high as 90 percent occurring during winter and spring rains. Floodflow slopes in the hill area range from 5 to 60 ft. per mile. The maximum flow of record is estimated as 33,000 c.f.s. at Valley Hill, the western edge of the hill area. The average annual precipitation is 52 in. The maximum hourly rainfall at Greenwood is 2.7 in., and the maximum 24-hour amount is 11.8 in.

Range Profiles

During a drainage survey for local interests in 1920, the Elliott and Harman Engineering Co. of Peoria, Ill., obtained ground profiles in a north-south direction on four section lines in the delta area of Big Sand Creek. Figure 5 shows the location of these profiles. During field surveys in 1947 for the Greenwood Protection Works, the Vicksburg Engineer District resurveyed these ranges. Figure 6 shows the deposition between the 1920 and 1947 surveys. These deposits were, to a large extent, the result of sediment dropped by crevasse flows through numerous and widespread breaks in the Big Sand levees near the hill line during this period. The four ranges were profiled again in 1957. The 1957 profiles are not shown in figure 6, since they did not differ greatly from the 1947 survey.

Sediment Deposits

Table 1 gives the amounts of sediment deposits on the cone area as computed from the range profiles and field investigations. Amounts

TABLE 1.—*Sediment deposits on cone area*

Time and location	Total	Sediment deposits per year		Per year per square mile
	Cu. yd.	*Cu. yd.*	*Acre-feet*	*Acre-feet*
1920–47 (27 years):				
Upper area	13,151,000	487,000	302	2.75
Lower area	12,254,000	454,000	281	2.55
Total	25,405,000	941,000	583	5.30
1947–57 (10 years):				
Upper area	810,000	81,000	50	0.45
Lower area	0	0	0	0
Total	810,000	81,000	50	0.45

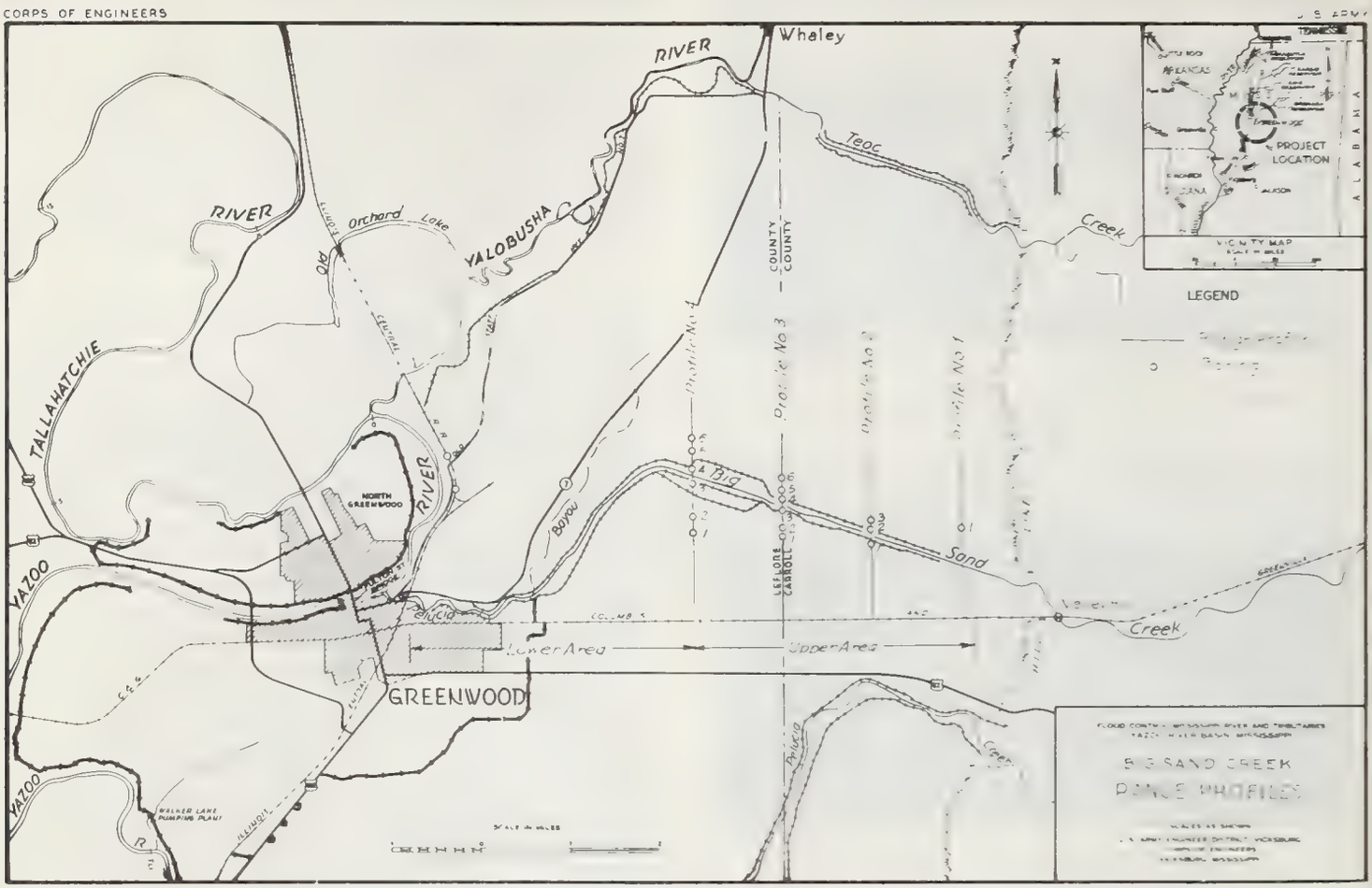


FIGURE 5. — Location of range profiles in delta area of Big Sand Creek.

of sediment deposits shown in table 1 for the upper area were computed directly from the range profiles. Amounts for the lower area were computed from depths of deposits obtained by field reconnaissance. Deposits on the cone are composed mainly of fine sands, silts, and clays. No estimates of deposits in the lower area were obtained in 1957; however, deposits in this area for the period 1947–57 are considered to be minor.

A major part of the difference between the rates of deposits on the cone area for the two periods is attributed to the improved maintenance of the levee system. During the latter period crevasses were limited in extent and were closed as soon as possible, thereby preventing long periods of flow through the crevasses and thus most of the sediment was transported to the Yazoo River. Evidence of this is indicated by comparative thalweg profiles of the Yazoo River in figure 7, which show considerable fill between 1940 and 1962. However, Big Sand Creek is only one of many small tributaries that, in addition to the major tributaries, contribute to the sediment load of the Yazoo River.

Estimates made in connection with the Mississippi River and Tributaries Project Review by the Soil Conservation Service in a report for the Mississippi River Commission entitled, "Report on Present and Anticipated Floodwater and Sedimentation Conditions, Yazoo Headwater Project, Part II, Big Sand Creek," dated August 1957, agree closely with the estimates made by the Vicksburg District from actual measurements. The estimates were based on empirical formula, analytical computations, and experience. The following tabulation shows the source of sediment and annual delivery rate to the delta area of Big Sand Creek as estimated in the Soil Conservation Service report.

Source [1]	Estimated gross erosion	Estimated delivered to delta area
	Acre-ft./yr.	*Acre-ft./yr.*
Sheet	648.5	129.7
Gully and roads	331.8	199.1
Streambank	392.3	392.3
Total	1,372.6	721.1

[1] Does not include any material from streambed since channel beds are believed in equilibrium.

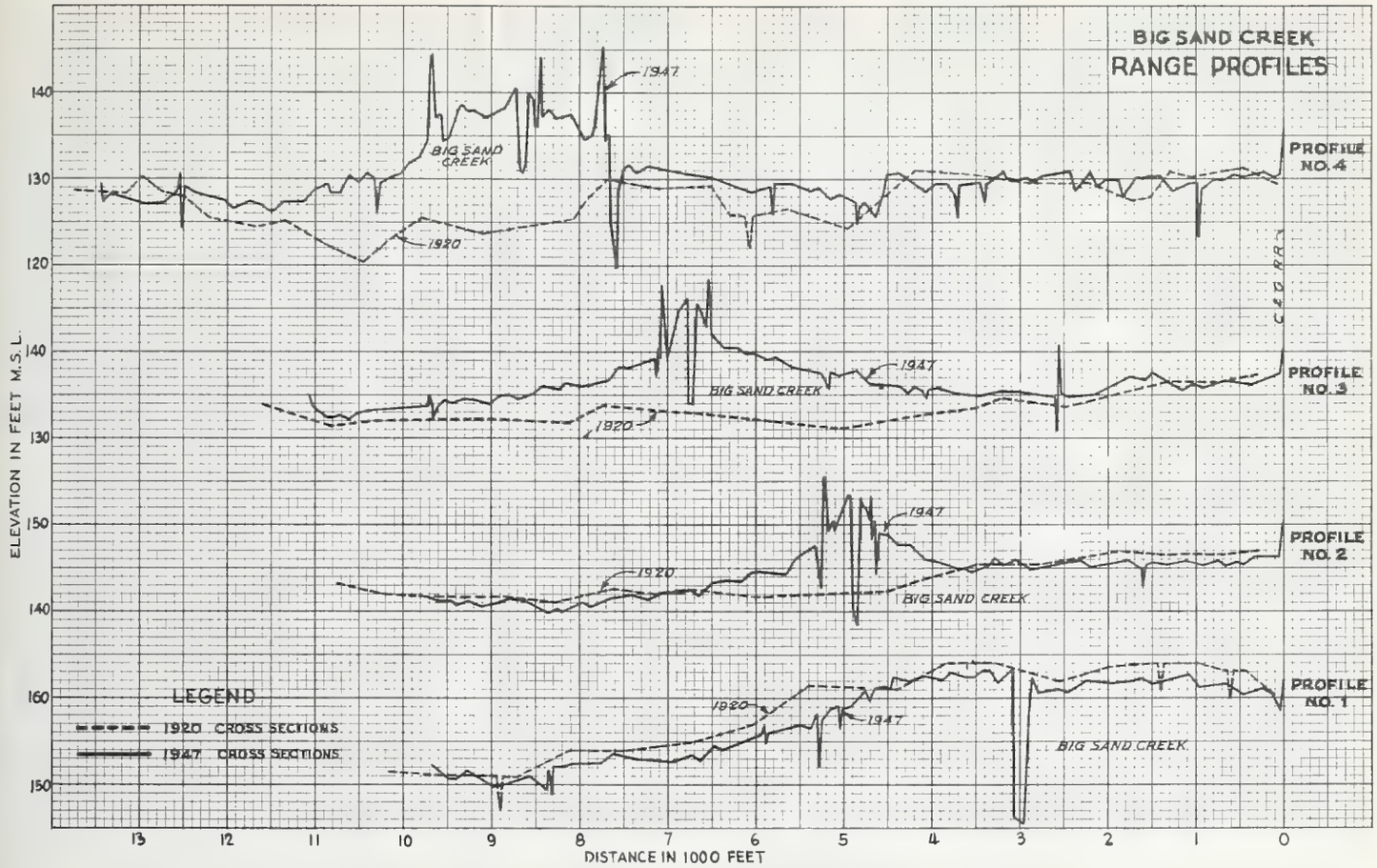


FIGURE 6. — Range profiles of Big Sand Creek, 1920 and 1947.

Summary

Big Sand Creek is one of the few areas where it was possible to obtain actual measurements of deposits on the ground as a basis for estimates of sediment loads. Although the estimates obtained are not highly accurate, they are reasonable and certainly serve to indicate the magnitude of the problem. The reduced rate of deposit in the cone area during the 1947–57 period is attributable to the fact that levees were maintained to reduce crevasse flow greatly and the opportunity for deposit in the cone area. Thus, a far greater part of the sediment load was transported to the Yazoo River than was the case during the earlier period. This has contributed to the filling in the Yazoo River in the 50-mile reach below Greenwood, as shown in figure 7.

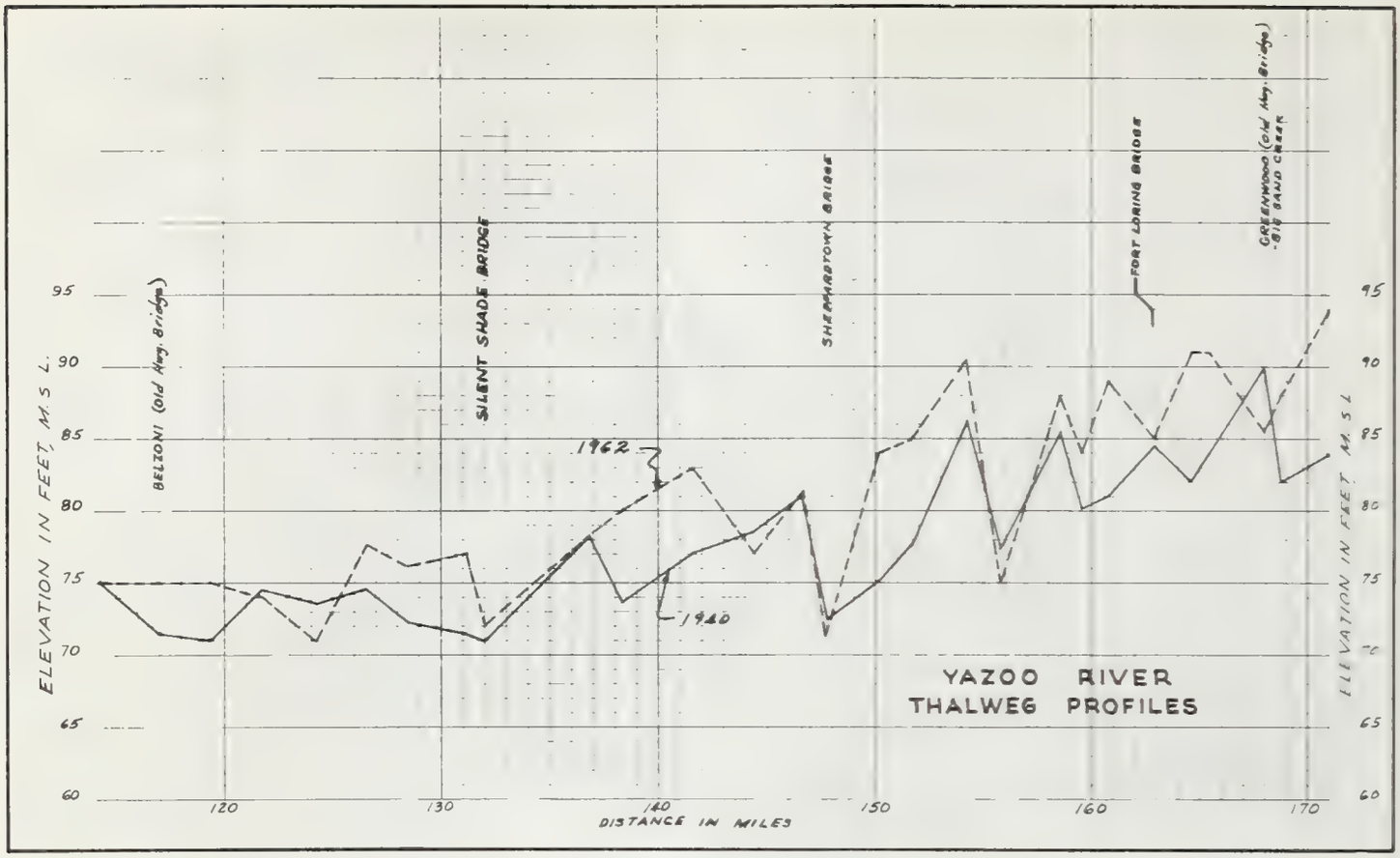


FIGURE 7. — Thalweg profiles of the Yazoo River, 1940 and 1962.

IMPROVEMENT OF THE NAVIGABILITY OF THE COLUMBIA RIVER BY DREDGING AND CONSTRICTION WORKS

[Paper No. 53]

By GEORGE E. HYDE, *chief*, and OGDEN BEEMAN, *supervisory construction engineer, Navigation Branch, Operations Division, U.S. Army Engineer District, Portland*

Synopsis

The Columbia River navigation channel from its entrance at the Pacific Ocean to the Portland-Vancouver area is maintained to dimensions of 35 by 500 feet by the removal of 12.2 million cubic yards of sediment each year. Constant improvement of the navigability of this 100-mile waterway has been made by the construction of permeable groins, removal of sediment by hopper and pipeline dredges, and use of dredge spoils for river constriction works. The controlling depth of the river was 12 feet in 1885; it is now 36 feet.

This paper reviews some work performed between 1957 and 1961 without giving details of earlier works on the river. The information is factual, not theoretical, and is based on hydrographic surveys of the river. The use of permeable groins and dredged sand in redirecting the river and improving channel depths on one bar is reviewed. The use of advance maintenance dredging to guarantee the availability of project depth all year, rather than just after completion of dredging, is considered. The success of this program in obtaining greater channel depths on a year-round basis is reviewed.

Several problems bearing on the maintenance of this navigation project are presented.

Introduction

The Lower Columbia River navigation channel from its mouth at the Pacific Ocean to the mouth of the Willamette River in the Portland-Vancouver area is an authorized project 35 feet deep by 500 feet wide. The channel is 98.5 miles long and dredging of approximately 9,800,000 cubic yards of river sediment is required annually through 26 river bars whose total length approximates 50 miles. In addition to this, an average of 2,400,000 cubic yards is

dredged from the Columbia River entrance bar each year. However, the term "river sediment" is slightly misleading, as the material dredged from the lower Columbia River consists almost entirely of fine to medium sand with very little silt to no clay. Absolute density of the material averages 2,700 grams per liter. The average length of the 26 river bars is 8,200 feet; the shortest is 2,000 feet and the longest is 14,000 feet. The other 48.5 miles of the channel under discussion is self-maintaining to river depths greater than 35 feet for a width of at least 500 feet. The river in its natural state had a controlling depth of 12 feet at St. Helens Bar (mile 86) in 1885.

River Constriction Works

It is recognized that the shoaling pattern on any river bar is a result of the interrelation of many factors. Principal among these factors in the Columbia River are:

(1) Characteristics of the annual river runoff, or "freshet," and its effect on the amount of bank erosion upstream.

(2) Action of river constriction works upstream and their tendency to pass material onto the bar being investigated.

(3) Action of river constriction works on the subject bar and the resultant shape and area of the river cross section.

(4) Placement of the navigation channel in relation to the thalweg of the natural river.

(5) Placement of the channel in relation to bends in the river.

(6) Type of sediment being transported by the stream.

It is not intended to present information where all the above variables have been held constant. However, by holding certain of these variables constant and changing others, a radical change in the channel over several years undoubtedly will be attributable to recognizable causes.

For purposes of comparison of shoaling, the years 1957 and 1961 were used because of the similarity of the flood pattern. Figure 1 shows the river hydrograph at Vancouver, Wash., for these 2 years. For purposes of comparison, since the freshets were similar, factor (1) mentioned above can be considered to have been constant when comparing shoaling in 1957 and 1961. No constriction works were installed for a distance of 20 miles upstream of St. Helens Bar between 1957 and 1961, so factor (2) was considered to be constant. Since the channel alinement was not changed and there is little change in the nature of the material from year to year, factors (4), (5), and (6) were likewise constant. Therefore, any change in shoaling on St. Helens Bar between 1957 and 1961 is primarily attributable to the change in river cross section caused by the addition of river constriction works.

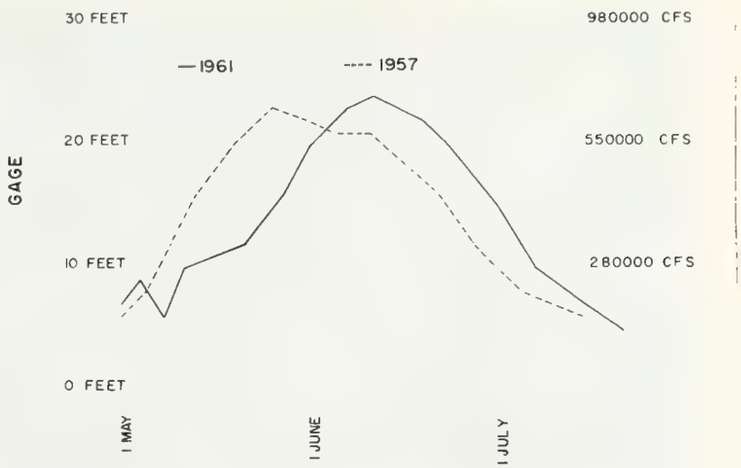


FIGURE 1. — Comparative flood-stage-discharge hydrographs, Columbia River at Vancouver, Wash., for 1957 and 1961.

Figures 2 and 3 are aerial photographs of St. Helens Bar, taken in 1957 and 1961. Through the years several permeable groins (i.e., pile dikes) have been constructed on this bar. In 1959 three additional groins totaling over 1,000 lineal feet were constructed, and in 1959 and 1960 hydraulic fill totaling 1,250,000 cubic yards was placed to supplement the permeable groins on the right bank in redirecting the river. Although this sand closure was breached the following year (see fig. 3), the effect of it on the navigation channel has been considerable. In 1957 the infill of sand onto this bar during the freshet was 856,000 cubic yards; in 1961, it was 364,000 cubic yards. It is believed this reduction in infill was a direct result of constructing the additional length of permeable groins and placing the 1,250,000 cubic yards of sand over and between these groins in the intervening years. As can be seen from the aerial photographs, the river cross section at this point was narrowed considerably and the river diverted completely by the groins and fill rather than partially diverted, as was the case with permeable groins only. Unfortunately, the hydraulic fill was only raised to elevation 22 feet and was topped by the freshet in 1961. It was breached in several places but was still effective in 1962. These breaches will be filled and the entire closure raised to flood-free elevation as material is available from maintenance dredging.

Slaughters Bar at mile 66 is 20 miles below St. Helens Bar. The intermediate bars between these two contain 30 permeable groins located so that a great deal of material is believed to pass through these bars. Slaughters Bar represents the natural settlement place for some of

FIGURE 2. — St. Helens Bar, Columbia River, November 1957.

the material because its cross section is too wide to support a 35- by 500-foot channel. As shown in figure 4, comparison of a naturally maintained section at the head of this bar at mile 67.0 with a middle section at mile 66.0 with shoals above 35 feet discloses that at least 300 feet of excessive width should be eliminated from the section at mile 66.0 to make it self-maintaining. If this were accomplished, the increased velocities generated thereby would cause the bottom to scour to a pattern similar to that shown for mile 67.0. However, due to strong objections from local interests, it has not been feasible to narrow this section by creating a middle ground or island built from dredge spoils and stabilized by permeable groins. While bars above Slaughters Bar are improving each year, it is significant that the trend on Slaughters Bar is toward worsening conditions. Table 1 demonstrates the trend of shoaling of a section of this bar.

It can be seen that as other reaches of the river are improved by constriction works, it may have an adverse effect on the navigability

TABLE 1.—*Shoaling on a section of Slaughters Bar*

Year	Shoal area after freshet [1]	Dredging depth	Infill into channel during freshet
	Sq. ft.	*Ft.*	*Cu. yd.*
1957 [2]	258,000	37	225,000
1958	710,000	37	[3]
1959	1,050,000	37	[3]
1960	1,347,000	37	[3]
1961 [2]	1,577,000	37	320,000

[1] Meaning total area within the navigation channel that is shallower than 35 feet.
[2] Similar freshet pattern.
[3] Not calculated.

of unimproved areas downstream subject to shoaling. It is assumed this is due to the increased inflow of bedload. It is not known if a reliable quantitative relation can be established between the amount passing through an upstream bar and the amount of material deposited on a lower bar under the conditions present in the Columbia River. The important question here is whether or not an increased bedload made up of material passing through upstream bars requires a much greater velocity to keep it moving. In other words, a bedload equal to X

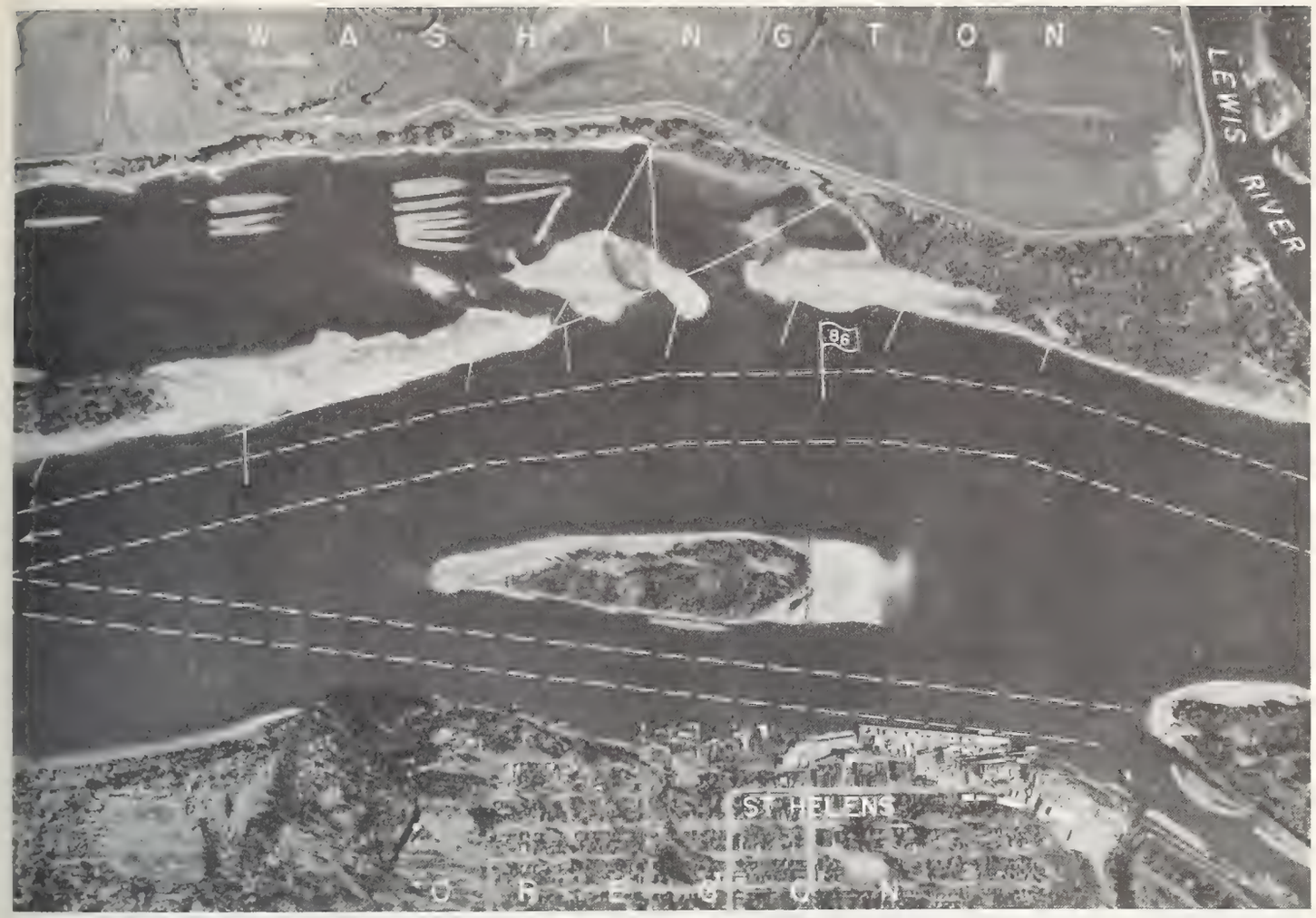


FIGURE 3. — St. Helens Bar, Columbia River, November 1961.

is passed through a bar by action of velocity V. When X is increased, will V necessarily have to be increased to prevent further shoaling on the downstream bar? The answer to this is not clear for the Columbia River.

Advance Maintenance Dredging

Experience indicates that nearly all of the shoaling on the Columbia River takes place dur-

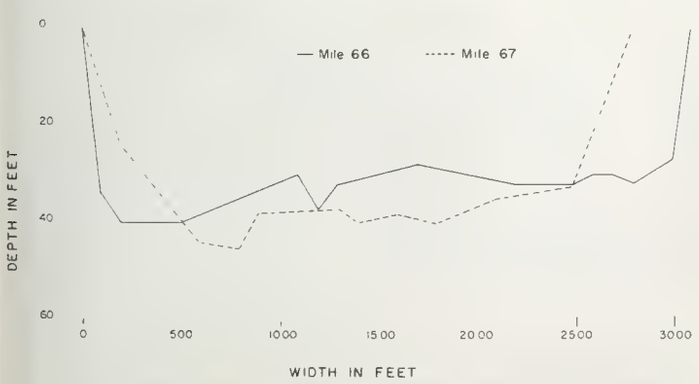


FIGURE 4.—Comparison of river cross sections at middle and head of Slaughters Bar, Columbia River.

ing the several weeks of the spring runoff or freshet. This means that after the river level returns to normal there is nearly 50 miles of river shoaled above project depth. This presents a critical situation from the standpoint of navigation, as the deeper draft ships are forced to delay arrival and sailing times so high tides can be utilized. During low river stages tides affect the project channel approximately 8 feet at the seaward end and 2.5 feet at Portland. Dredging capacity of the district is taxed immediately following the freshet in order to deepen the channel in the shortest possible time.

It has been customary to dredge project depth plus 2 feet allowable overdepth, or to 37-foot depth on the Lower Columbia River. Maximum shoaling during the average freshet is 6 to 8 feet, leaving a controlling depth of approximately 30 feet on some bars. So that the channel may be at project depth the year around and in order that dredging may be scheduled on a year-around basis, Portland District, during the last several years, has endeavored to perform what is called advance maintenance

dredging. This means that rather than dredging to 37 feet, the channel is excavated to a depth of 40 or 42 feet; i.e., project depth, plus 3 to 5 feet of advance maintenance dredging plus 2 feet of allowable overdepth dredging, to allow for the inaccuracies of the dredging process.

It can be seen under this plan that after 6 to 8 feet of infill, there would still be left the project depth of 35 feet, and all shoaling would have taken place below project depth. The advantages of this plan are obvious. A good example of this practice is maintenance of Willow Bar at river mile 97. Table 2 shows results of the advance maintenance program from 1957 to 1961, inclusive: Table 3 shows conditions at bars after freshets of 1957 and 1961.

TABLE 2.—*Willow Bar maintenance by advance dredging*

Year	Dredging depth previous year	Shoal after freshet	Controlling depth at centerline	Number of soundings at depth [1]					
				29	30	31	32	33	34
	Feet	*Sq. ft.*	*Feet*						
1957[2]	37	1,205,000	31–	6	4	12	17	33	47
1958	37	1,835,000	31–	7	5	16	19	51	84
1959	40	335,000	31–	1	6	3	4	11	22
1960	42	44,000	35	0	0	0	0	0	2
1961[2]	42	48,000	34	0	0	0	0	1	7
1962	40	197,000	33	0	0	0	1	6	9

[1] Each sounding represents approximately 7,500 sq. ft.
[2] Similar flood patterns.

TABLE 3.—*Columbia River bars—comparison of condition after freshets of 1957 and 1961*

Bar	1957		1961		Remarks [1]
	Shoal area	Controlling depth on centerline	Shoal area	Controlling depth on centerline	
	1,000 sq. ft.	*Ft.*	*1,000 sq. ft.*	*Ft.*	
1. Desdemona	200	35	480	36	
2. Flavel	100	35	150	37	
3. Upper Sands	690	34	60	35	
4. Tongue Point	610	33	330	34	Bars 1–9 are hopper dredged. Shoal areas may remain after dredging due to inaccuracies of the dredging method.
5. Miller Sands	1,810	33	2,080	33½	
6. Pillar Rock	180	35	325	30	
7. Brookfield	710	33	900	31	
8. Skamokawa	490	35	280	32½	
9. Puget Island	250	35½	200	35	
10. Wauna	2,080	30½	680	35	Dredged to −40 feet in 1960.
11. Westport	710	33	0	38	Dredged to −42 feet in 1960.
12. Eureka	750	29½	150	33	Do.
13. Gull Island	200	34	120	37	
14 Stella-Fisher	3,350	29½	420	36	Dredged to −42 feet in 1960.
15 Walker Island	1,420	29½	340	36	
16. Slaughters	1,930	33	2,920	31½	
17. Lower Dobelbower	1,540	31	310	35	Dikes extended in 1959.
18. Upper Dobelbower	1,390	31	310	31½	Do.
19. Kalama	2,120	31	750	33½	New dike in 1960; also dredged to −42 feet in 1960.
20. Lower Martin Island	120	35	70	34	
21. Upper Martin Island	700	33½	1,380	29	
22. St. Helens	2,240	30	100	39	New dikes in 1959; also dredged to −42 feet in 1960.
23. Warrior Rock	140	37	160	36	
24. Henrici	780	34	1,320	30½	Not dredged in 1960.
25. Willow	1,210	31	50	34	Dredged to −42 feet in 1960.
26. Morgan	120	32½	120	35	Not dredged in 1960.
Average	994	32.8	539	34.2	Reflects effect of both dikes and advance maintenance dredging.

[1] All bars from Nos. 10 to 26 were dredged to 37 feet the previous year unless otherwise noted. Bars 1 to 9 were dredged to a maximum of 37 feet with minor shoals of 34 feet remaining at the completion of dredging.

It is apparent from the preceding tabulations that advance maintenance dredging accomplishes its objective of making greater depths available on a year-around basis. Although it is recognized that there is an obligation to maintain this channel to authorized depth, it is of interest to compute the benefit-to-cost ratio of making the increased depths available. It is obvious that overdepth must be originally dredged and its cost amortized over the life of

the project. Calculations show that approximately 10 million cubic yards of advance maintenance dredging would need to be accomplished initially so that all shoaling would take place below project depth. Based on an assumed cost of 22.5 cents per cubic yard and on a proposed life of 30 years, this dredging would cost about $75,000 per year (no interest included). In addition to this, it seems reasonable that annual dredging costs would increase due to the greater shoaling taking place; i.e., because velocity equals flow/area, it follows that an increase in cross section will bring about increased shoaling due to the decreased velocities. Means of estimating this increased shoaling are not presently available for this project. It has been determined that, in general, 5 feet of advance maintenance dredging will increase the total river cross section at a typical bar by about 5 percent. For purposes of economic evaluation, it is assumed that the shoaling will be increased by that amount.

Experience shows that an expenditure of $1,800,000 (1957 dollars) is required to keep the channel to depth of 35 feet for 215 days per year and to depths of 29 to 31 feet for 150 days per year. The additional cost to keep the channel to 35 feet all year is as follows:

(1) Initial cost amortized at 30 years, with no interest:

10,000,000 cubic yards at 22.5 cents per cu. yd. = $75,000

(2) Increased annual dredging costs:

5 percent × $1,800,000 = $90,000

Total increased annual cost to obtain 35-foot project depth year around..... $165,000

It is assumed that various increments of this sum would maintain channel depths as follows:

Depth all year (feet)	*Annual cost of dredging in excess of $1,800,000*
30	$0
31	33,000
32	66,000
33	99,000
34	132,000
35	165,000

The determination of the benefits to navigation largely depends on how much time deep draft ships are required to wait for the tides to aid in traversing the project during the 150 days after the freshet. No detailed study was made of subsidiary benefits, such as eliminating the necessity to stop off in other ports, as the reason for the study was only to ascertain whether advance maintenance dredging is economically justified.

On the Columbia and Lower Willamette Rivers project, deep-draft vessel traffic during the 5 months in calendar year 1961 when project depth was not available, was as follows:

Draft at stern of vessel:	*Approximate number of vessel trips during 5-month period*
33–34	5
32–31	151
30–29	173
28–27	160

As was stated above, controlling depth is 30 feet for 5 months per year when only $1,800,000 is spent for dredging.

It is further estimated that under these conditions each vessel noted above loses 3 hours per trip awaiting a favorable tide during the 5-month period. It is also assumed that reduced ship draft of 2 feet for squat and 2 feet for safety were suffered. If this wait of 3 hours, as developed above, is eliminated, benefits will accrue as shown in table 4.

TABLE 4.—*Benefits accrued from maintaining channel depths of Columbia River*

Draft, including squat and safety factor (Feet)	Vessels in 5 months	Awaiting favorable tide, hours at 3 hours per vessel	Benefits (hours×$200)[1]
	Number	*Hours*	
31 and greater plus 4 feet	156	468	$93,600
29 plus 4	173	519	103,800
27 plus 4	160	480	96,000
Total dollars			293,400

[1] Vessel hour assumed to cost $200.

Figure 5 shows these values plotted. Also plotted is the cost to attain these various depths as described above. Dredging advance maintenance at all depths up to 35 feet is justified. The benefit/cost ratio to maintain 35 feet all year is 293,400/165,000 or 1.78:1.

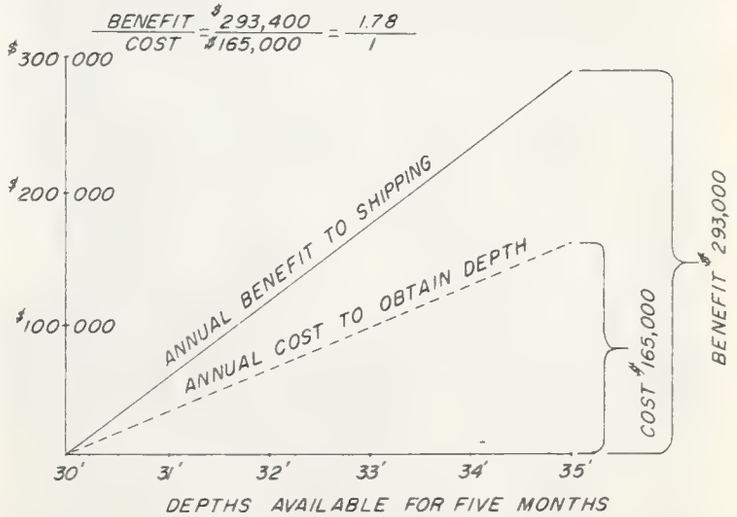


FIGURE 5.—Benefit cost ratio to obtain 35-foot depth year around.

Advance maintenance dredging in addition to the benefits to navigation is advantageous. A large pipeline dredge requires a relatively deep infill bank for efficient dredging. Figure 6 demonstrates the reason for this. To assure an efficient dredging program, the pipeline dredge should work against the heaviest possible bank.

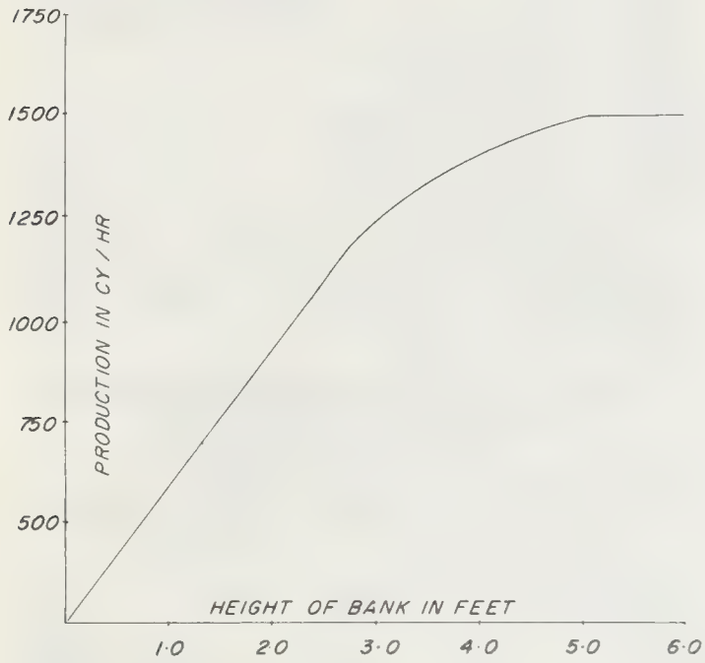


FIGURE 6.—Production curve for 30-inch-pipeline dredge *Clackamas* pumping through 2,000-3,000 feet of discharge line, Columbia River.

One means of doing this is to dredge advance maintenance and allow bars to shoal for 1 or more years. That is, a bar which normally shoals 2 to 3 feet per year could be dredged to 6 feet of overdepth every other year. The cross section would be slightly increased, which would in turn increase the tendency of the bar to infill the following year. However, dredge production may be increased greatly or at least enough to more than offset this increased shoaling. An increase of dredging bank from 3 feet to 6 feet increases production of the dredge *Clackamas* from 1,250 to 1,550 cubic yards per hour (fig. 6). Since dredge cost is virtually constant, regardless of production, this would reduce unit costs by 24 percent.

From the standpoint of dredging efficiency, it is unfortunate that shoaling does not take place by consistent and level buildup on river bars. The shoaling is mostly in sand waves and, therefore, a 32-foot sounding may be followed closely by one of 42 feet. This is clearly shown in the reproduction of a section from a fathogram shown in figure 7. It is obvious that this situation does not lend itself to pipeline dredging. The trailing suction hopper dredge is one method of eliminating these waves. It is believed that, if available, trailing suction hopper dredges can be utilized to great advantage to remove the scattered high spots in a channel. This allows the rest of the bar to build up toward a heavier bank, which in turn assures later efficient use of pipeline dredges.

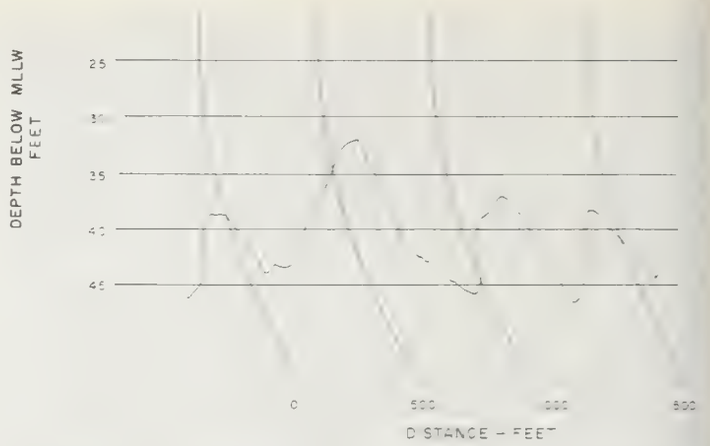


FIGURE 7.—Fathogram along centerline navigation channel, showing sand wave shoaling.

Use of River Sediment

We tend to lose sight of the fact that river sediment can be and in many places is a most valuable natural resource. Effectual usage of dredged sand depends upon the location and the foresight and planning of the engineers dealing with the sedimentation problem. In the Columbia River, dredge spoils are utilized for river control and constriction works and for development of recreational areas.

The use of sand for river control and constriction works certainly represents the highest and best use of sediment on the Columbia River project. The use of sand for constriction works has generally been in conjunction with the permeable groin building program. Permeable groins of untreated piling are used almost exclusively for river training in this area. Several permeable training groins have been built on river bends, where permeable spur groins are not sufficient to turn the river.

Normally, in constructing these groins, two rows of piling are driven with staggered pattern, so the effective spacing is 2½ feet. The two rows are separated by a waler which is securely bolted to all piling. The cutoff elevation varies between +6 and +10 feet on Columbia River datum. A rock blanket is placed after the piling is driven to protect the structure from scour. Because of the permeability of these groins, it is almost always desirable to fill between the groins with dredge spoils to supplement natural accretion. The ultimate effect is like that of a long training groin following the desired curvature of the river and moving the

bank line channelward. This is believed to be the most economical solution to the problem of dredge spoil disposal. However, when the spur groin method is used on sharp bends, excessive scouring and erosion of the toe of the slope of the sand fill presents a problem.

Constricting the river by placing sand fill without the benefit of spur groins has been fairly successful in some areas. However, to be fully effective, sufficient sand must be available to bring the fill to floodfree elevation and must be placed continuously channelward from normal high waterline. Even then, the toe of slope and the side of slope is endangered during subsequent freshets. The fill at St. Helens Bar (fig. 3) is thought to be a good example of the combined use of permeable groins and dredge spoils to obtain river control.

The use of river sand for recreational uses has been limited to establishing beaches or nourishing existing beaches for use by fishermen, picnickers, and boaters. Cooperation with State, county, and municipal groups has resulted in numerous public fishing and recreational beaches being created in past years in those reaches where use of dredge spoils is adaptable to this activity.

Conclusion

The Portland District has made great strides in improving the navigability of the Columbia River during the past years. River constriction structures and the use of dredge spoils as well as advance maintenance dredging have brought about a marked improvement to the ship channel.

There are several major problems whose solution is not readily apparent but must be accomplished to insure continued improvement of the Columbia River navigation channel. These problems are:

1. Determination of a low-cost method for removing the peaks from the sand waves that develop in the ship channel and interfere with navigation.
2. A sound, harmless method of determining percentages of material that reenters the navigation channel under various dredging disposal methods to allow logical derivation of costs of various dredging disposal procedures.
3. Determination of how much shoaling increases when a channel is overdredged in depth but river width remains constant.

Solution of these problems will enable the Portland District to continue improvement in the navigability of the lower Columbia River.

CHANNEL ARMORING BELOW FORT RANDALL DAM

[Paper No. 54]

By Robert H. Livesey, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

One of the major problems encountered in the design of outlet facilities for reservoir projects, particularly those planned for alluvial streams, is the determination of tailwater levels for optimum operating conditions. Generally the degradation estimates are based upon various methods of analytical study involving numerous related variables, many of which have not as yet been formulated into adequate mathematical terms, or upon engineering judgment derived from observation and study of experienced conditions. The variables in this problem include the transport relation for the fine riverbed material, the amount of coarse material available to armor the bed, the presence of such fixed degradation controls as rock outcrops and clay ledges, the pattern of reservoir releases, and the degree to which the banks are stabilized to prevent the channel from widening or meandering.

This paper is directed toward the phenomenon of channel bed armoring. It describes results that occurred from the segregation of a small quantity of gravel on the surface of the Missouri River streambed below Fort Randall Dam in South Dakota. Extensive sampling of the riverbed surface, plus deep exploration of the subsurface strata of both the channel and the overbank areas, prior to construction of the dam, indicated only a small percentage of gravels down to the probable lower limit of degradation. Yet a one-grain thickness of gravel has accumulated over the entire bed surface for a distance of several miles, overlaying erodible sands, to halt significantly the desired degradation trend. This paper describes the natural characteristics of the Missouri River reach below Fort Randall Dam and conditions after 10 years of degradation. Detailed photographs of the bed armoring layers will be of interest. The tailwater degradation trend is presented. The armor layers and degradation trends obtained in one series of flume studies compare with the observations below Fort Randall.

Introduction

Seldom does an engineer engaged in hydraulic design work enjoy the latitude of having a complete and factual picture of the basic criteria

necessary for the proper analysis of a problem. When the analysis encompasses any sedimentation phenomena, particularly on alluvial streams, the problem immediately becomes further magnified due to the many interrelations that exist between numerous variables, which to date have been neither adequately identified nor formulated into usable mathematic terms. In addition, even adequate knowledge of several of the basic equilibrium factors is somewhat vague and unpredictable, or perhaps indiscernible altogether. To the young or inexperienced engineer these limitations might seem a ponderous block that necessitates extensive research and study of the many facets before attempting to resolve the problem. However, the answers necessary to establish design criteria for proposed projects cannot wait. They must be based upon the best analytical methods available at the time and upon judgment derived from the observation and study of experienced conditions.

This paper is focused upon the problems encountered in the design of outlet facilities for the Fort Randall Reservoir project on the Missouri River in South Dakota and specifically upon the phenomenon of the channel bed armoring that occurred. At Fort Randall the determination of tailwater levels for optimum operating conditions included an evaluation of future degradation trends and probable limits. Variables in this problem included the transport relation for the fine riverbed material, the amount of coarse material available to armor the bed, the presence of such fixed degradation controls as rock outcrops and clay ledges, the pattern of reservoir releases, and the degree to which the banks are stabilized to prevent the channel from widening or meandering.

The final design of the powerhouse and outlet works are based upon a detailed analytical study, which was supervised by eminent consultants in the sedimentation field and included extensive but limited field investigation surveys. This study indicated that an ultimate 15-foot lowering of the tailwater levels would occur from the expected regulation of flows. Since the closure of Fort Randall Dam in July 1952, the total tailwater lowering experienced to date has been only about 3.5 feet, with 2 feet occurring during the first 2 years. A small layer of gravel, one grain in thickness and overlying the erodible sands, has segregated out of the degrading bed and accumulated upon the bed surface for a distance of several miles to cause this significant reduction in the desired degradation trend. The following description of the characteristics of the Missouri River in this reach plus an account of the degradation phenomena observed since closure of Fort Randall Dam in 1952 may be of general interest to the engineer faced with the problem of predicting degradation.

The Fort Randall Dam and Reservoir project is located on the Missouri River in the south central part of South Dakota (fig. 1). It is operated as one unit of a comprehensive plan of development of the Missouri River basin. The reservoir has a capacity of 6,300,000 acre-feet and extends 111 river miles upstream to the Big Bend Dam and Reservoir project. Diversion of the river through the outlet works was accomplished in July 1952. Storage to modify significantly downstream discharges began in about June 1953.

Missouri River Below Fort Randall Prior to Closure of the Dam

The Missouri River below the Fort Randall Dam site is confined to a narrow valley by chalk bluffs spaced fairly uniformly about 1.5 miles apart. The valley is alluvial, with the channel bed composed primarily of sand that is constantly in a state of movement. Gravel pockets exist at infrequent and irregular intervals in the underlying stratum, with occasional boulder outcrops in the vicinity of local valley wall drains. The channel banks and valley floor are composed of sandy silt with occasional areas of clay that were deposited in overbank pockets during floods. The river channel itself is alined in a semimeandering shape, with an alternating series of straight reaches adjacent to the chalk bluffs and intermediate crossing reaches where the channel shifts diagonally across the valley to the opposite bluff contact. The channel averages about 2,000 feet in width and is generally divided into more than one channel by large wooded islands or low, shallow sandbars. Due to the extensive bluff contacts in this particular reach of the river, the channel alinement has not changed materially since 1892.

The natural flow pattern in this reach was characterized by a definite annual pattern. During the winter months the flows were greatly reduced, due to freezing weather and the accumulation of ice that eventually formed a solid cover over most of the reach. Usually in March or April the warmer temperatures induced surface runoff of the uplands snow pack, which caused ice breakups to occur. Normally this spring runoff would be relatively moderate in volume, but frequent ice jams would produce extensive flooding and record maximum stages. After the spring breakup the river would subside to below bankfull stages and maintain an irregular pattern dependent upon the frequency of the spring rains. A second rise, usually greater in volume, occurred about the first of June and was governed by the availability of runoff from mountain snowmelt. Frequently this rise

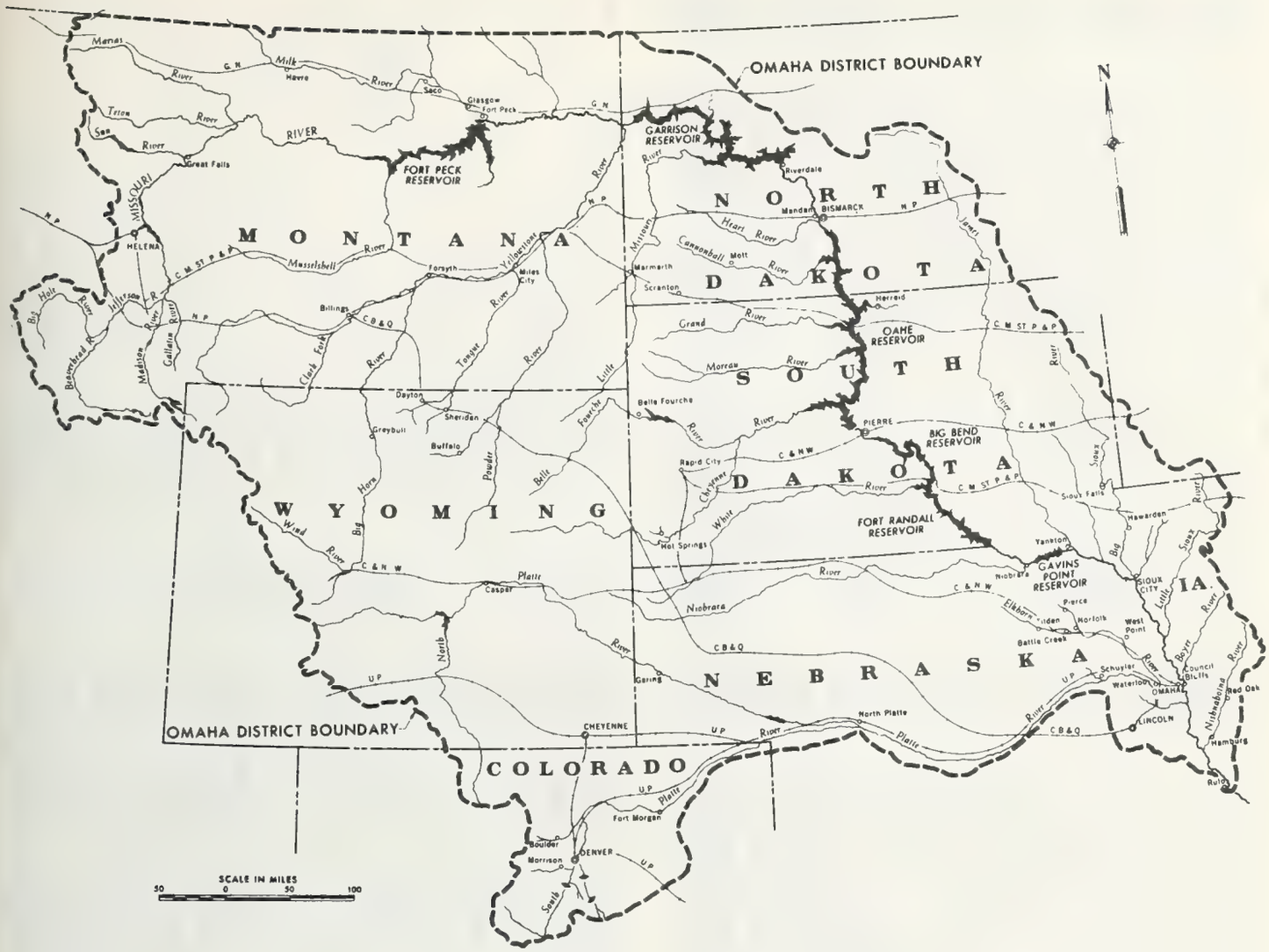


FIGURE 1. — The Omaha District and Fort Randall Reservoir.

was augmented by severe early summer thunderstorms. After the recession of this June rise, the flows continued at a reasonably steady but slowly diminishing rate through the late summer and fall until freezeup again occurred to repeat the cycle. The maximum flood of record at Fort Randall was 448,000 c.f.s., which occurred in April 1952, just prior to closure of the dam. The normal summer flows generally ranged from 16,000 to 70,000 c.f.s., and the winter flows between 9,000 and 15,000 c.f.s.

The general shape of the Missouri River channel, below the valley flood plain, is rectangular with an average width of 2,000 feet and bank heights varying between 8 and 15 feet. The natural channel has a bankfull capacity of about 150,000 c.f.s. and an average gradient of 0.8 foot per mile. At normal summer flow stages the active channel width varied between 700 and 1,500 feet and the mean depth between 4 and 11 feet. Greater widths and shallower depths occurred in the diagonal crossings between opposite bluff contacts.

The Missouri River has long been noted for its capability to transport a large and variable quantity of suspended sediments. It is estimated that the average annual sediment load transported past the Fort Randall Dam site was 148 million tons for the period 1898–1952. Based upon the particle-size distribution observed from 1943 to 1952 at Yankton, S. Dak., it is estimated that 30 percent of this annual load was sand, 30 percent silts, and 40 percent clays. The material composing the streambed surface of the natural channel consisted of fairly uniform, fine sand with a median size of 0.20 mm. A small percentage of larger material, generally well rounded glacial particles varying in size up to 1 inch, could be found in the deep holes at the downstream ends of submerged bars.

The available data on the original bed composition of the Missouri River near Fort Randall

are summarized by the mechanical analysis curves in figure 2. Curve 1 on this figure, based

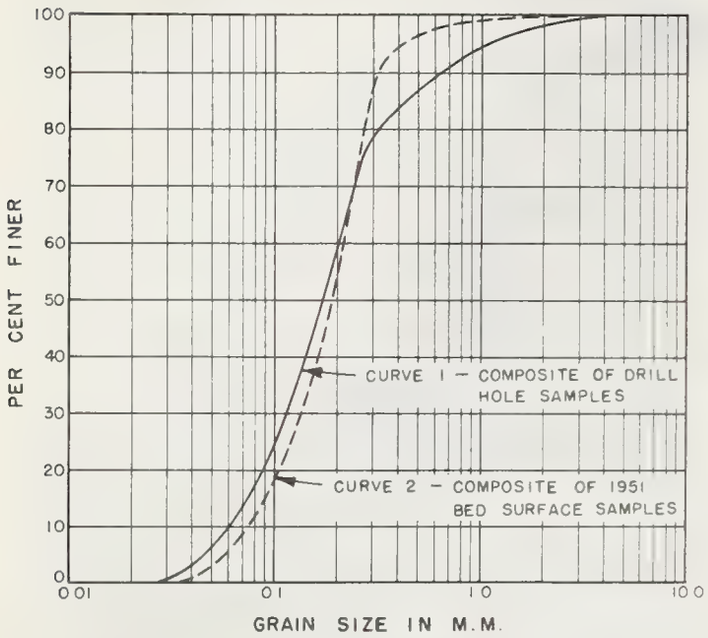


FIGURE 2. — Grain-size distribution of available bed materials as determined from drill hole and bed surface sampling.

upon drill hole samples, indicates the mean composition of the first 10 feet or so of material beneath the bed surface. Curve 2 is based on bed surface samples. The two curves up to the 80 percent finer size are similar, but above this point curve 1 shows significantly more coarse material than curve 2 for the bed surface. Applying a little hindsight at this point, it appears these curves may be somewhat misleading in that they probably show less coarse material than actually existed in the undisturbed conditions. For example, the BM–48 bed material sampler employed in obtaining many of the bed surface samples tended to be selective, due to its inability to always penetrate a gravel bed surface and retrieve the larger gravel particles. Second, neither the BM–48 nor the Straub bed material samplers are capable of obtaining samples that represent only the composition of particles forming the bed surface. These samplers bring up not only the surface layer of particles but also about one-tenth of a foot of the material underneath. Third, some gravel particles were probably lost from the churn drill samples between the bottom of the drill hole and the deck of the drill barge.

Reservoir Operation

The pattern of monthly average releases from Fort Randall Dam is shown in figure 3. The

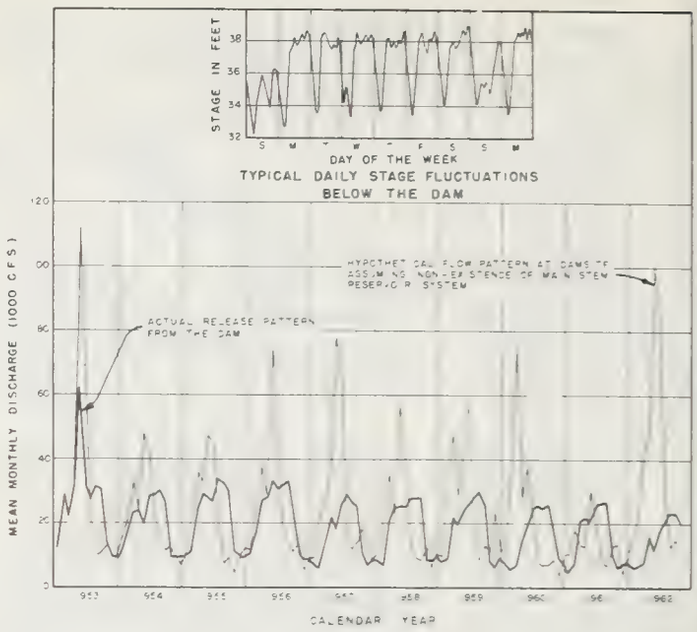

STAGE IN FEET
DAY OF THE WEEK
TYPICAL DAILY STAGE FLUCTUATIONS BELOW THE DAM
MEAN MONTHLY DISCHARGE (1000 C F S.)
ACTUAL RELEASE PATTERN FROM THE DAM
CALENDAR YEAR

FIGURE 3.—Actual and hypothetical release trends from Fort Randall Dam.

regulatory effect of the reservoir system can be assessed by comparing the lower curve, which shows the actual releases, with the upper curve, which estimates the flows that would have occurred with no regulation. It can be seen that since 1953 the average monthly releases have followed a persistent pattern. During the navigation season the average release has been generally in the 20,000 to 30,000 c.f.s. range. During the winter months these flows were curtailed to 5,000 to 10,000 c.f.s. These flow patterns, however, do not portray the entire picture. Initial power generation from Fort Randall was started in 1954, and by 1956 all normal releases were being made through the eight turbines. Since this power project has been operated primarily as a "peaking plant," the resultant releases normally have a daily fluctuation from a low of nearly zero to a high of more than 40,000 c.f.s. during the navigation season and over 20,000 c.f.s. during the winter. Stage fluctuations resulting from the daily release pattern for a typical week during the navigation season are shown in the insert on figure 3. These power peaking operations have resulted in daily stage fluctuations up to 7 feet at the dam, decreasing to 5 feet at the U.S.G.S. measurement station 7 miles below the dam and 4 feet at Greenwood, located 15 miles below the dam.

Effects of Ten Years of Operation

For the first 2-year period after closure, the tailwater levels lowered at a rate of 1 foot per year, but since 1954 the annual rate has diminished to about one-tenth of that value (fig. 4). As also noted on figure 4, the initial degradation

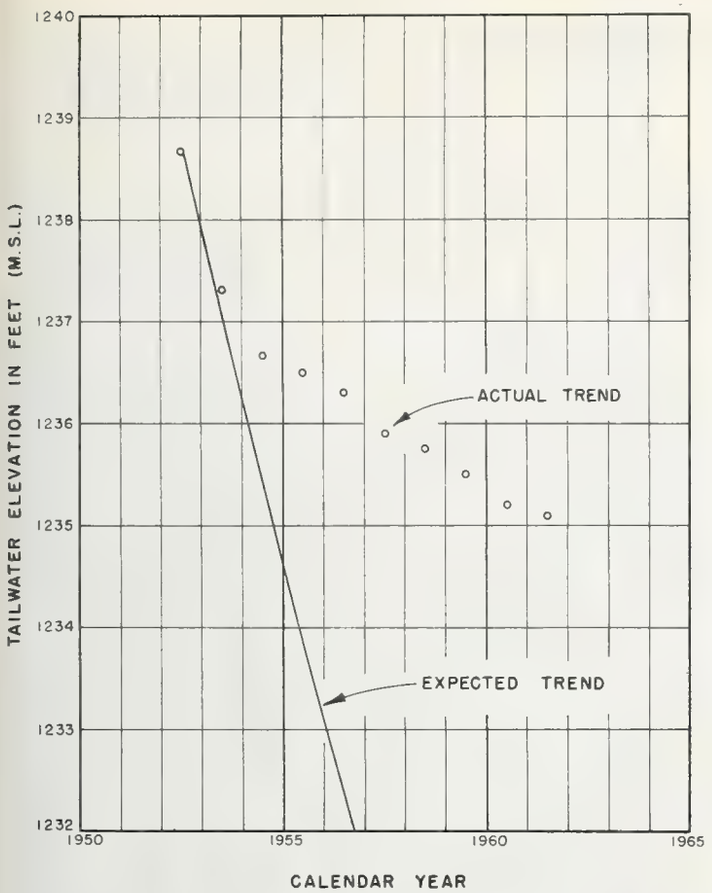


FIGURE 4. — Tailwater elevation trend since closure of Fort Randall Dam in 1952.

trend approximated the expected rate very closely until 1954. Comparative rating curve data at a U.S.G.S. measurement station 7 miles downstream from the dam indicated a major shift in 1952, due to the maximum flood of record, but only minor yearly changes since dam closure.

Frequent cross sectional resurveys were made at the degradation ranges below the dam to record volumetric changes. In order to evaluate properly the observed changes, however, the configuration of the river channel must also be considered. From the dam downstream to about river mile 919 the channel maintains a fairly uniform width of about 2,000 feet along a left bank bluff contact, while between river mile 919 and 915 it makes a diagonal valley crossing to another bluff contact on the opposite bank. Within this reach the channel widens to more than 3,000 feet, while the river meanders in two or more threads of flow between low sandbars. Below river mile 915 for the next 3 miles it again assumes a bluff contact while repeating another diagonal valley crossing between river mile 912 and 909.

Restricting our analysis to this upper 15 miles of the degradation reach, we record on figures 5 and 6 comparative cross sectional area, width, average bed, and thalweg changes as referenced to a common plane that paralleled the natural water surface profile and intersected most sections at about a bankfull stage. Thus, the data reflect changes occurring not only within the wetted perimeter but also to the bank lines of the channel and larger sandbar islands.

As might be expected under degrading conditions the channel widths tend to increase; again this widening occurred, particularly in the wide crossing reaches. Most of this apparent widen-

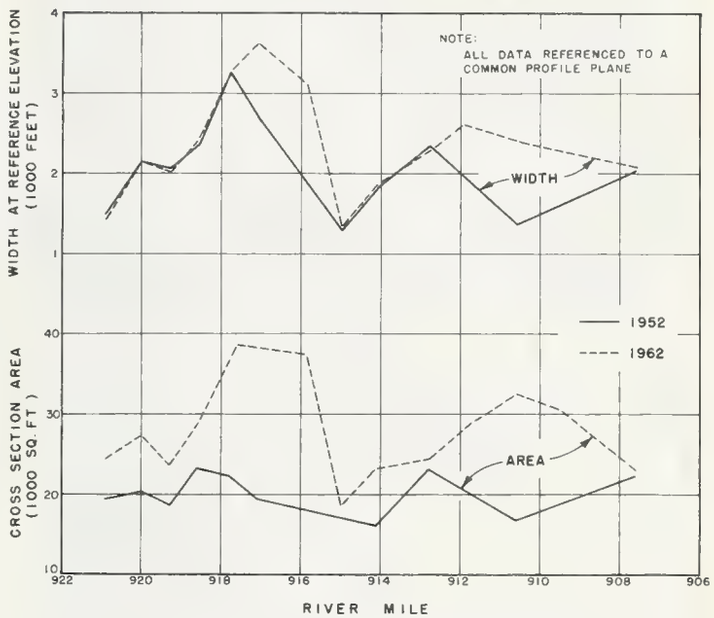


FIGURE 5. — Changes in cross sectional width and area as observed at degradation ranges between 1952 and 1962.

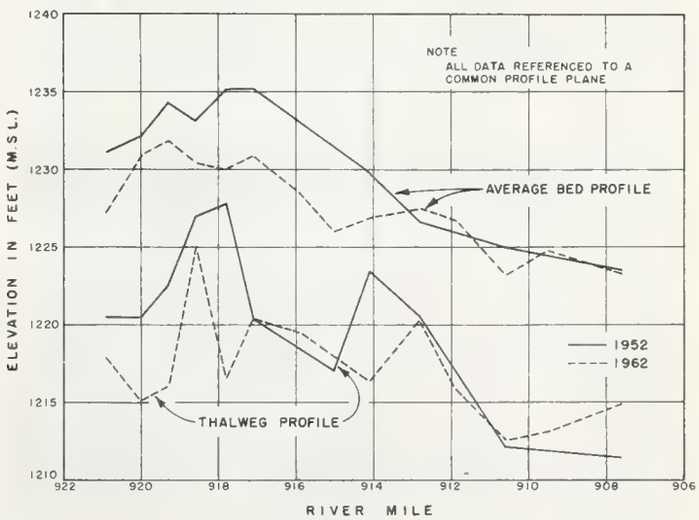


FIGURE 6. — Changes in average bed and channel thalweg elevations as observed at degradation ranges between 1952 and 1962.

ing, however, has been due to the disintegration of the channel sandbars; only a secondary amount of widening has been due to erosion of the high channel banks. For example, at river mile 917.1 the channel width increased 928 feet in the 10-year period, yet only one-third of this change was true bank erosion. The rest was loss of sandbars within the channel between the high banks. The flows attacked the bars in preference to the equally erodible channel banks. The plot of the cross section areas and the associated average bed profile also exhibit considerable changes, but again this represents predominately the influence of the reduction in sandbar area and not cutting of the high banks. In the lower crossing reach below river mile 912, the major change was due to extensive bank erosion; however, this was the result of a natural shift in the channel alinement that had been developing since the flood of 1952.

The volume of material that has been transported from this reach, as determined by end area computations, also reflects the trends noted in cross sectional area and width changes (fig. 7). Again, the major differences are noted in

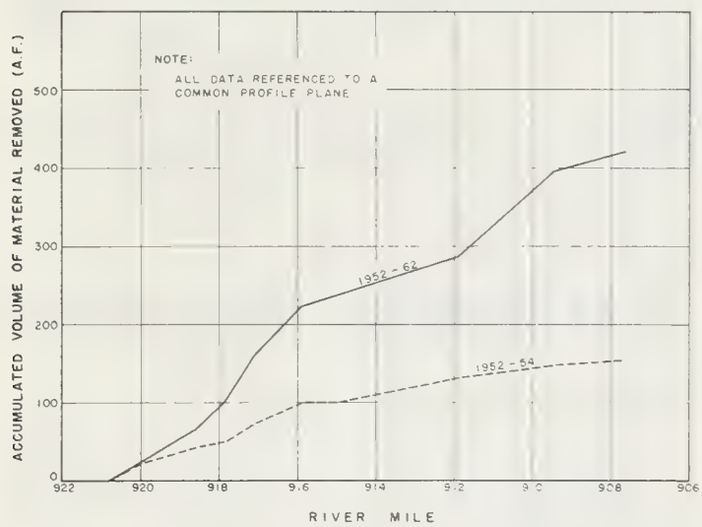


FIGURE 7. — Estimated volume of material removed by natural stream transport from the upper Fort Randall degradation reach between 1952 and 1962.

the channel crossing reaches where bed and bank materials are being introduced into active transport. Although the total volume removed approximates an average of 1,000 acre-feet per year, this value converted to an average daily sediment transport concentration represents less than 100 parts per million, as compared with a daily average of about 5,000 parts per million in the natural river prior to construction of the dam.

The changes in bed surface composition since 1951 are illustrated in figure 8, where the D_{90}, D_{65}, D_{35}, and D_{10} grain sizes are plotted against river mile for the years 1951, 1954, and 1960. There has been a significant coarsening of grain size throughout the reach from the dam downstream to river mile 907. For example, D_{65} was originally about 0.20 mm. In 1960 it was near 1.0 mm. at the dam, decreased progressively to about 0.35 mm. downstream to river mile 914, and remained fairly uniform at about this size below that point. D_{90} and D_{35} exhibit similar patterns. The uniform bed composition downstream of river mile 914, considered in conjunction with figure 6, which exhibits little change in bed elevations in this same reach, leads to the conclusion that this reach is temporarily in equilibrium, with the equilibrium load being supplied by degradation upstream of river mile 914. Degradation can be expected to progress downstream in years to come. Why there does not seem to be a significant change in bed surface composition throughout the reach between 1954 and 1960 is not readily explained. Perhaps such differences would be apparent if

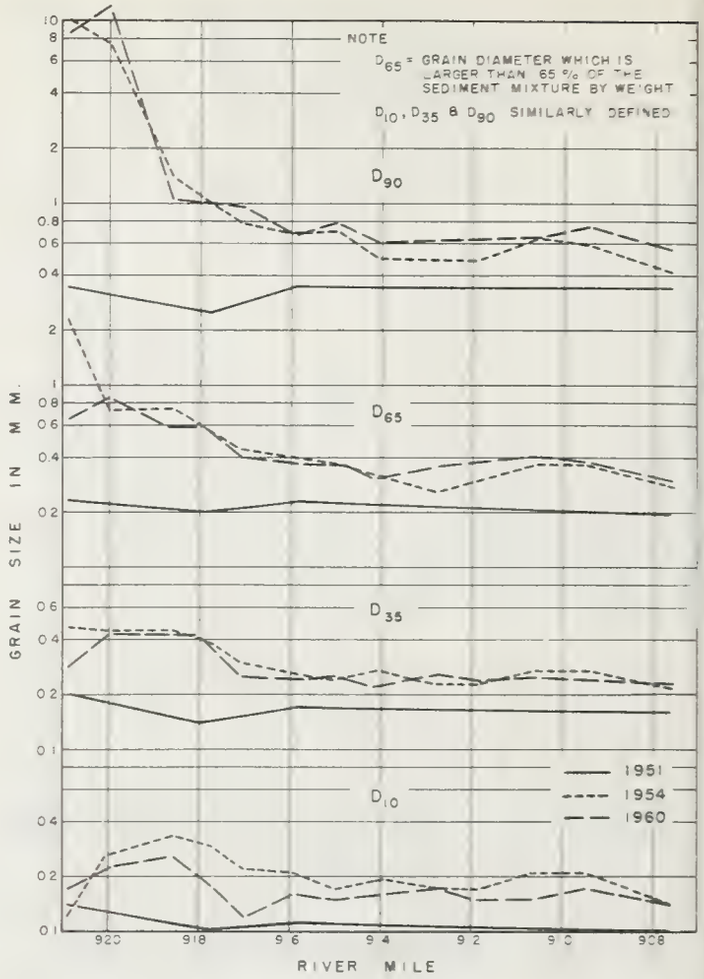


FIGURE 8. — Changes in the grain-size distribution of the streambed surface material between 1951 and 1960.

the actual bed surface layer were sampled instead of the top 0.05 to 0.10 foot of the bed as discussed previously.

Armoring

The coarsening of the bed surface is associated with the phenomenon of armoring. As the sediment transporting capacity of the clear water released from Fort Randall Dam tended to become satisfied by scouring material from the riverbed, a continuous net lowering of the bed surface occurred, first near the dam, and progressing downstream. At the same time, due to the regulatory effect of Fort Randall Reservoir, both the incidence and the duration of high discharges were drastically reduced when releases were maintained in the 10,000 to 30,000 c.f.s. range after 1952. Curve 2 on figure 2, based on bed surface samples obtained under normal discharge conditions, indicates that few particles larger than about 1 millimeter are normally exposed at the bed surface. This would indicate there was normally little movement of grains larger than this size. Curve 1, based on the underlying bed material, however, indicates about 5 percent of the subsurface material is larger than 1 millimeter. In the natural river these larger grains were probably transported only during the annual flood periods when the discharge usually ranged above 100,000 c.f.s. The degradation trend, the regulation of flows to reduce the incidence of high discharges, and the presence in the bed of a fraction of material that is relatively nonmoving at normal discharges, were the elements necessary for armoring to occur.

The armoring process begins with a prolonged period of regulated flows during which the nonmoving particles segregate from the finer transport and are worked down into the bed where they accumulate in a sublayer at the lowest level to which the bed is turned over by the bar movement that accompanies the transport process. This lowest level is about in the plane of the troughs that occur just downstream of the crests of the moving bars. As bed material is transported from the reach below the dam, the entire bed system, the bars as well as the coarse sublayer, degrades. In this process fine bed material is leached up through the coarse sublayer to augment the material in transport. Bar movement continues, but as degradation progresses, an increasing number of nonmoving particles accumulate in the sublayer. Soon this accumulation interferes with the leaching process so that the rate of transport over the sublayer is not maintained at its former intensity. The moving bars begin to shorten with increasingly greater areas of the coarse sublayer exposed between them. Eventually enough coarse particles accumulate to shield, or "armor," the entire bed surface. Fines can no longer be leached from the underlying bed, transport in the reach is halted, and degradation is arrested. Figure 9 illustrates this armoring process in a flume study conducted at the University of California by A. S. Harrison.[1] A more dramatic illustration of the characteristics of armor layers has been presented in movies taken by L. G. Straub at the St. Anthony Falls Hydraulic Laboratory during experimental studies of gravel blankets for streambed stabilization for the Arkansas River project of the Corps of Engineers.[2]

This same armoring process — the gradual transition from continuous bar movements to the stable bed surface — developed below Fort Randall Dam; but observations were limited to sampling and soundings until the spring of 1962 when high inflow conditions on the lower river necessitated a lengthy period of zero releases from the dam. This circumstance allowed the channel to drain to a very low stage into a series of pools and permitted a visual inspection of a part of the streambed surface. The photographs shown as figures 10 through 14 represent typical channel bed conditions as observed through the armored reach, which extended several miles below the dam.

A general view of the bed surface and a typical examination hole is shown in figure 10. The characteristic shingled arrangement of the surface particles should be compared with the texture of the surface obtained by Mr. Harrison in his flume study (fig. 9). Figure 11 shows a closeup view of the cross section of an examination hole. Attention is directed here to the thickness of the armor layer and the composition of the underlying material, including an isolated particle of coarse gravel 6 inches below the surface. The large gravel particles, which appear so numerous in these photographs, were not detected from the original bed sampling and borings, probably for the reasons previously discussed.

The closeup views in figures 12 through 14 show in detail these important characteristics of the armor layer:

(1) That less than a single complete covering layer of larger gravel particles seems to suffice for a total armoring effect. Figure 12 shows a complete gradation of particle sizes throughout the gravel range.

(2) A natural "filter" seems to develop between the larger surface particles and the sub-

[1] HARRISON, A. S. REPORT ON SPECIAL INVESTIGATION OF BED SEDIMENT SEGREGATION IN A DEGRADING BED. Calif. Inst. Engin. Res., ser. 33, issue No. 1. September 1950.

[2] Presented at American Society of Civil Engineers Water Resources Engineering Conference at Omaha, Nebr., May 1962.

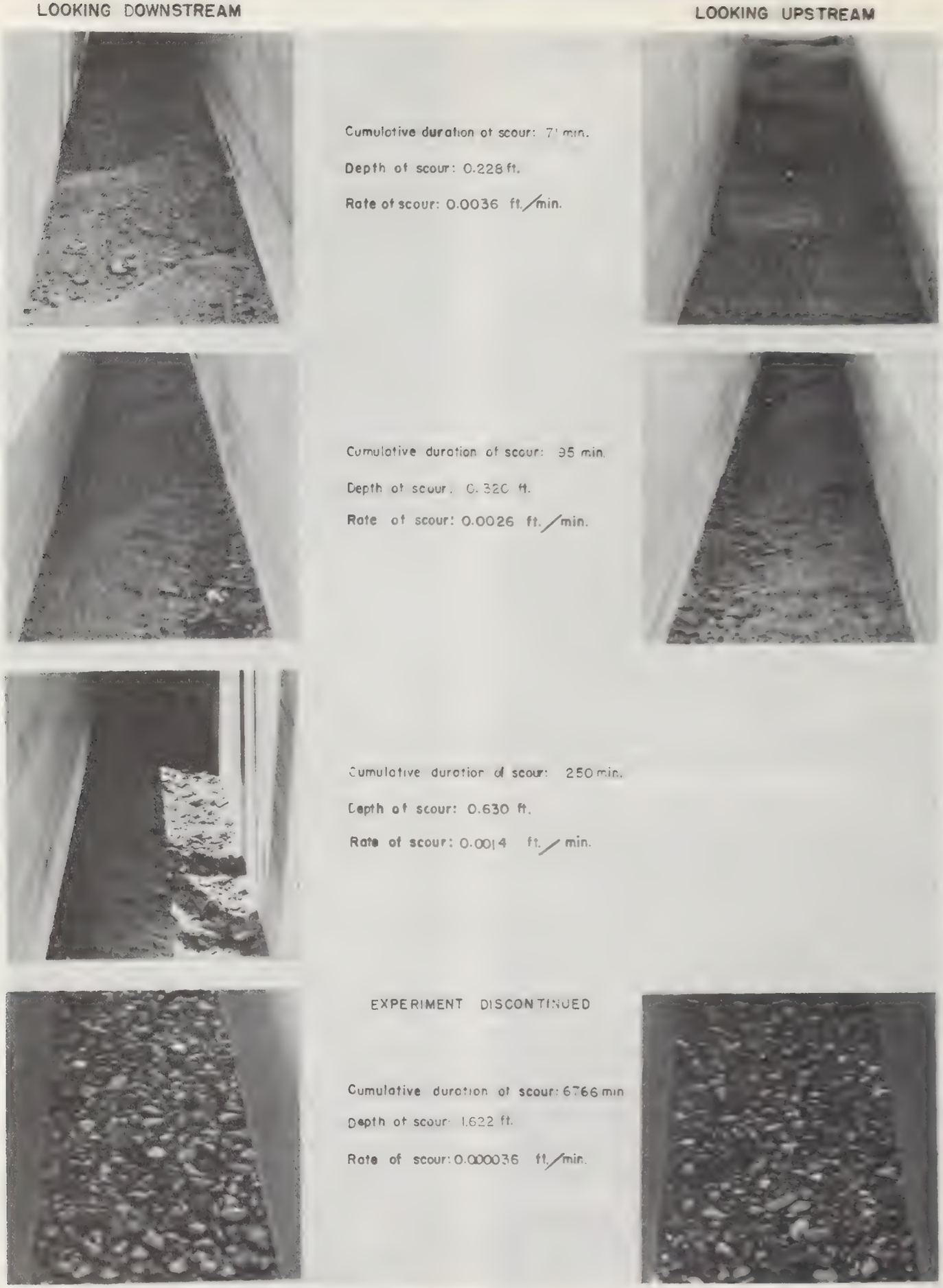

LOOKING DOWNSTREAM
LOOKING UPSTREAM
Cumulative duration of scour: 7' min.
Depth of scour: 0.228 ft.
Rate of scour: 0.0036 ft./min.
Cumulative duration of scour: 95 min.
Depth of scour. 0.320 ft.
Rate of scour: 0.0026 ft./min.
Cumulative duration of scour: 250 min.
Depth of scour: 0.630 ft.
Rate of scour: 0.0014 ft./min.
EXPERIMENT DISCONTINUED
Cumulative duration of scour: 6766 min
Depth of scour: 1.622 ft.
Rate of scour: 0.000036 ft./min.

FIGURE 9. — Changes in bed characteristics during scour, as observed by A. S. Harrison in a flume study conducted at the University of California.

surface material to prevent leaching of the underlying fines. Figures 13 and 14 demonstrate that a surface layer of large particles

FIGURE 10. — General view of bed surface armor below Fort Randall Dam. Observations were made during extremely low stages in 1962. Reference scale in the examination hole has a 6-inch length.

FIGURE 11. — Cross sectional closeup of the examination hole shown in figure 10. Note armor thickness and composition of underlying materials, including an isolated particle of coarse gravel about 6 inches below the bed surface.

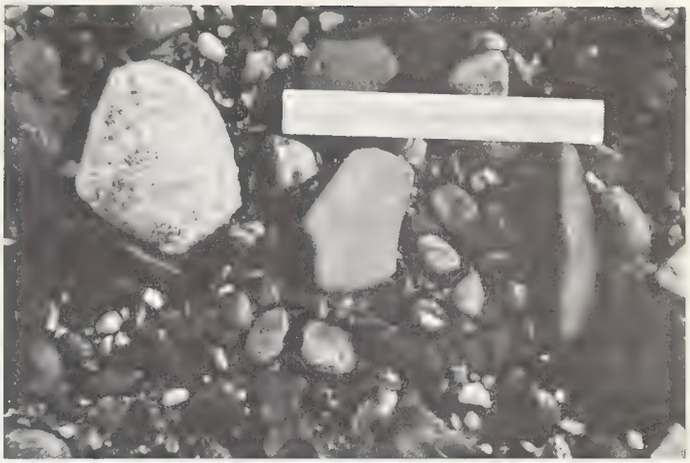


FIGURE 12. — Closeup view of typical bed surface armor. Note range of particle sizes.

FIGURE 13. — Cross sectional closeup view of streambed armor, showing isolated example of a concentration of coarse gravel in the underlying strata.

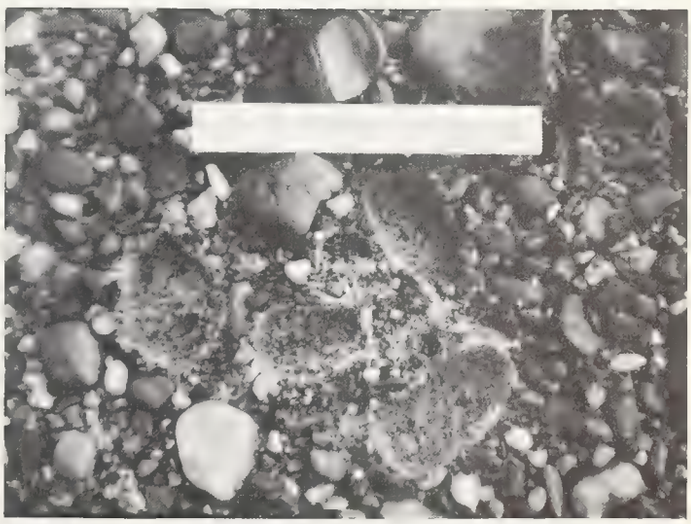


FIGURE 14. — Closeup view of bed surface armor, with several large gravel particles removed to show underlying sands.

does not have to be complete if medium particles are present to form a "filter" to fill in the interstices. The coarser sands collect beneath this surface.

(3) The shingled arrangement of surface particles is not restricted to the larger material but seems evident throughout the gravel gradation. This condition is very apparent in the arrangement of particles shown in figures 12 and 14.

If a single layer of material is sufficient to establish a bed armor, then armoring can develop quite readily. If a one-half inch particle is nonmoving and only 1 percent of the material underlying the bed surface is greater than one-half inch, then the depth necessary to accumulate a single surface layer of one-half inch particles would be 0.5 inch $\div$ 0.01 = 50 inches, or approximately 4 feet of degradation. Actually less degradation than this would suffice. First, the thickness of a single layer of alluvial gravel tends to be less than the nominal particle size, because the minor axes of the particles are smaller than the semimajor axes. The particles tend to lie with their minor axes vertical, although the semimajor axes determines the nominal sieve size of a particle. The second reason is that the bed need not be completely covered with armoring gravel if a filter layer of intermediate size particles is present between the armor layer and the underlying bed. For these reasons, it is probable that about a half layer of nonmoving particles is sufficient to armor the bed. This conclusion is supported by the results of the flume study cited above.

Conclusions

The rapid channel armoring which developed below Fort Randall Dam significantly arrested the desired degradation trend. It appears that three of the elements necessary for armoring to occur prevailed in this reach: (1) the degradation trend, (2) the regulation of flows to reduce the incidence of high discharges, and (3) the presence in the streambed of a fraction of material that is relatively nonmoving at normal discharges, plus a full gradation of material finer than this nonmoving particle size. Through visual inspection of the existing armor layer it becomes clear that less than a single layer of larger particles is sufficient for a total armoring effect, the armoring effect is particularly due to the natural filter that develops beneath the surface layer to prevent leaching of the finer underlying material. The shingling arrangement of the surface particles appear to reduce the maximum surface coverage necessary of the larger nonmoving particles. These characteristics of the bed armor below Fort Randall indicate that perhaps further degradation might be encouraged by disturbing this surface layer through artificial scarifying or jetting action.

STABILIZATION OF STREAMBEDS WITH SHEET PILING AND ROCK SILLS

[Paper No. 55]

By WALTER M. LINDER, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

The design of the Floyd River flood control project in Sioux City, Iowa, included a high velocity rock-lined channel that would carry the design discharge of 71,500 c.f.s. below the level of the adjacent natural ground. A high potential for erosion of the channel bed required the development of a series of channel stabilization structures that would maintain the desired water surface elevation and deter destructive bed degradation and upstream head cutting. A brief model study at the University of California at Berkeley indicated a row of sheet piling across the channel bed with some form of rock protection might adequately retard the development of head cutting and create sufficient losses to maintain the desired upstream water surface. Further model studies were conducted at the University of Iowa to develop criteria for adequate rock protection for the sheet piling and to obtain relationships for determining the head losses to be used for design purposes.

This paper summarizes the conditions that led to the development of the channel stabilization structure and describes the procedures followed in the model studies. The results of the model studies are discussed and presented in the form of curves suitable for general design purposes. Also suggested are some of the areas in which additional investigation would be profitable.

High Velocity Channel for the Floyd River Flood Control Project

The flood control project for the Floyd River consists of approximately 6 miles of realined and improved channel through the business and industrial area of Sioux City, Iowa. The design of the lower mile of the project was dictated to a great extent by severe limitations on the available right-of-way and by the numerous streets

and railroads traversing the area. Since the realined channel passes through an area occupied by numerous business and industrial establishments and extensive railroad yards, the right-of-way requirements had to be held at an absolute minimum. The numerous street and railroad crossings and adjacent railroad yards required that the water surface elevation be kept as low as possible in order to avoid the prohibitive cost of raising the railroad yards and adjacent street grades. Also the bed grade of the realined channel had to meet the bed grade of the existing natural channel at the head of the reach, and at the same time it could not be so steep as to produce excessive velocities for discharges up to 71,500 c.f.s. A general plan of the Floyd River flood control project is shown on figure 1.

Normally these stringent requirements would dictate the use of a high velocity concrete-lined channel; however, the cost of such a channel would have been prohibitive. The selected design calls for a trapezoidal channel with an erodible bed and rock protected 1 on 2½ side slopes. The bottom width of this channel narrows from approximately 280 feet at the Missouri River to about 100 feet at the upper end of the reach. Design water depth ranges from 17 to 25 feet from the lower to the upper ends of the reach, respectively. Average velocities through the reach will be slightly in excess of 14 ft./sec. for the design discharge of 71,500 c.f.s. Velocities will be greater than 10 ft./sec. for floods greater than 23,000 c.f.s., which is approximately the 25-year flood.

Throughout the rock-lined reach the channel lies in relatively erosion-resistant silts and clays of the Missouri River flood plain. However, a few feet below the design bed grade lie deep deposits of highly erodible sands and gravels. With the erodible material lying so close beneath the channel bottom there is a high potential for deep localized scour and extensive bed degradation with the occurrence of high velocity flows. In the event extensive degradation occurred, it could cause extensive damage by undermining the bridge piers and abutments and the toes of the revetted side slopes. Another consequence of degradation would be excessive drawdown of the water surface and destructive velocities in the leveed earth channel upstream of the rock-lined reach.

It was therefore necessary to develop a series of stabilization structures that would deter head cutting in the event degradation occurred and create sufficient head losses to maintain the desired water surface elevations. The preliminary design contemplated the driving of single rows of sheet piling across the channel at strategic locations with the tops of the sheet piling at or slightly below the design bed grade. At that time no specific information was available on the amount of head loss that could be expected once the sheet piling was exposed and projected into the flow. Also lacking was information as to its effectiveness in preventing head cutting and the amount of rock protection required to insure the stability of the sheet piling itself.

Model Studies at the University of California

At the time the high velocity channel for the Floyd River was being designed, model studies on rock sills for the Gering drain project were in progress at the hydraulic laboratories of the University of California at Berkeley. A brief two-dimensional study was set up to see what the behavior of a row of sheet piling across an erodible bed might be. The flume used for this study was a 60-foot long recirculating channel 2 feet wide and 1 foot deep. A piece of sheet metal across the channel and anchored to the flume bottom was used to represent the sheet piling. A 4-inch depth of small polystyrene pellets was used to represent an erodible bed. Since the 1.055 specific gravity of the pellets was only slightly greater than that of water, they moved at the slightest disturbance and served as an excellent indicator of the scour-inducing potential of the structure. No rock protection was used in the vicinity of the model structure.

The structure was tested at several different discharges with varying tailwater conditions and varying amounts of structure exposure above the average bed elevation. The results of the test indicated that sufficient losses could be generated over the sill, and that stability of the sheet piling did not appear to be a major problem. Based on the results of these studies, the initial design of a series of four sills for the Floyd River channel was completed. The sills consisted of a V-shaped row of sheet piling across the channel at a 10° angle with the normal and with the point of the V facing upstream. A 24-inch horizontal layer of rock was placed upstream of the sheet piling for a distance of 15 feet with the surface of the rock 3 feet below the top of the sheet piling. Downstream of the sheet piling a 36-inch layer of rock extended on a 1-on-2 slope from the top of the sheet piling to a depth of 9 feet below the top of the piling. The adjacent side slopes were protected with a 36-inch layer of rock, which tied into the horizontal blanket upstream and extended to a depth of 9 feet below the top of the piling on the downstream side of the structure. The original intention of the upstream facing V was to concentrate the flow over the sill into the center of the downstream channel and away from the

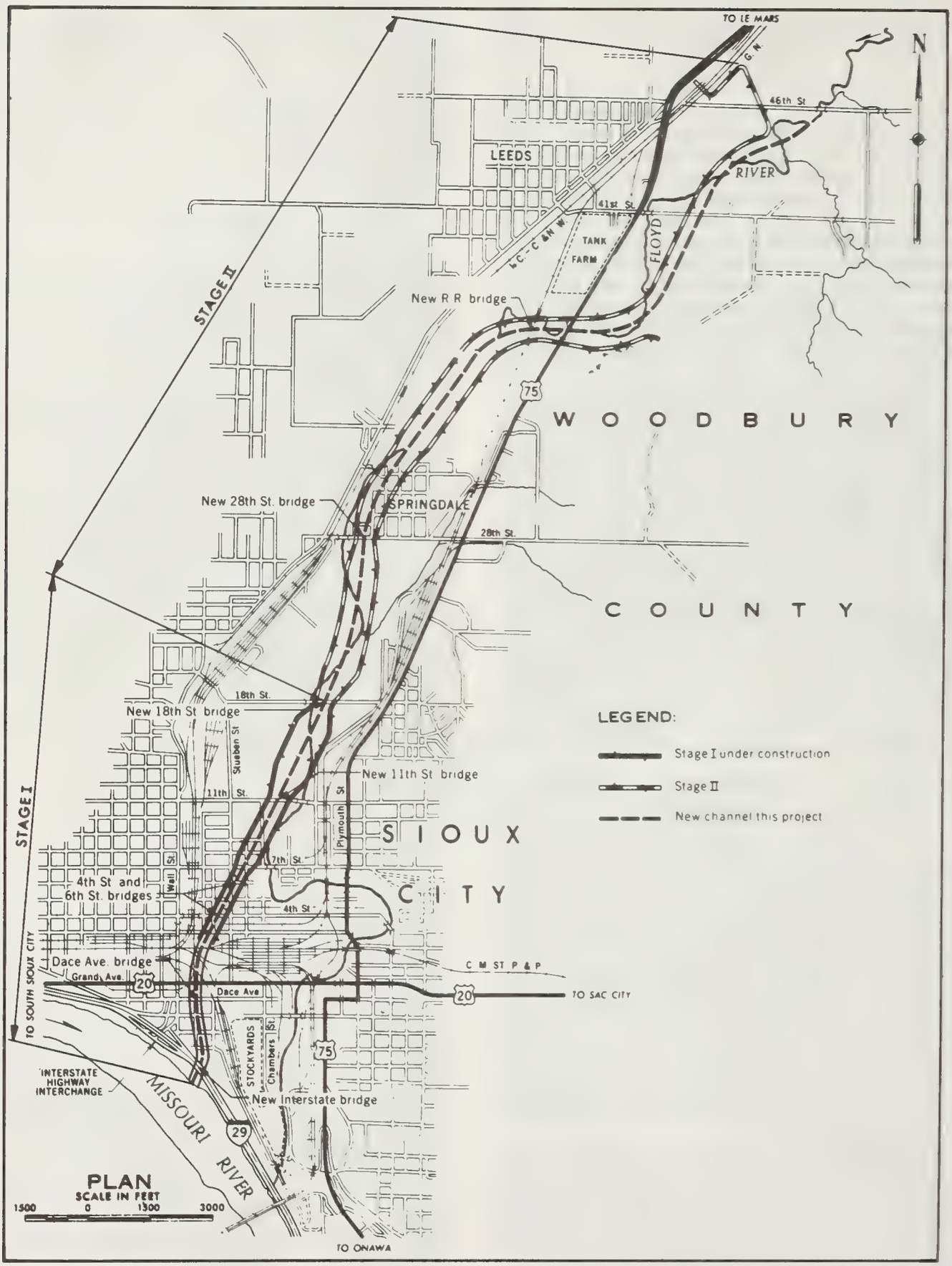


FIGURE 1. — General plan of the Floyd River Flood Control project.

side slopes, but model tests later showed that a straight transverse row of sheet piling gave better performance.

Model Studies at the University of Iowa

Objectives and Available Facilities

When the time came for the Iowa Natural Resources Council to give the required approval for the construction of the project, there was doubt expressed as to the ability of a series of sills to control channel erosion and prevent serious damage to the channel and related structures. Approval of the channel work was withheld until it could be demonstrated that the sills would perform as expected. In cooperation with the Iowa Natural Resources Council, an agreement was reached whereby personnel of the Omaha District, Corps of Engineers, would use the facilities of the hydraulics laboratory of the University of Iowa at Iowa City to construct a three-dimensional model of one of the proposed sills and demonstrate the performance of the structure. At the same time any deficiencies in the proposed design would be corrected; and, as time permitted, general design information would be obtained. Specific items of information desired were the minimum rock protection required, discharge coefficients, and the effect of submergence.

The flume used in these studies consisted of a concrete block channel 30 feet long, 5 feet wide, and 18 inches deep. The available discharge was approximately 5.25 c.f.s. Level rails on the walls of the flume permitted movement of a traveling carriage used for measurements of the water surface and the channel bed. The rails also served as guides for the screeds used in construction of the model channel. In addition, the flume was equipped with a mechanically driven sediment feed elevator at the upstream end and a combined sediment trap and weighing tank at the downstream end (figs. 2 and 3).

The 5-foot flume width permitted the use of a 1:50 scale ratio for the three-dimensional model. The relation between the model and prototype for this scale ratio, based on Froude's Law, are given in the following table:

	Prototype	*Model*
Channel bottom width...feet....	140	2.80
Channel length...feet.....	1,250	25
Design discharge...cubic feet per second	71,500	4.04
Average velocity...feet per second	14	1.98
Depth of flow...feet.....	25.3	0.505
Riprap, median size...inches in diameter	22	0.45
Derrick stone..inches in diameter	36 to 48	¾ to 1

Construction of the Model

The model bottom width of 2.80 ft. and depth of 0.505 ft. required a channel top width of 5.30 ft., or 0.30 foot wider than the flume. This resulted in a very slight reduction of the flow area. To avoid trouble with the intersection of the rough channel side slopes and the smooth flume walls, a strip of expanded metal screen was placed against the flume walls to provide

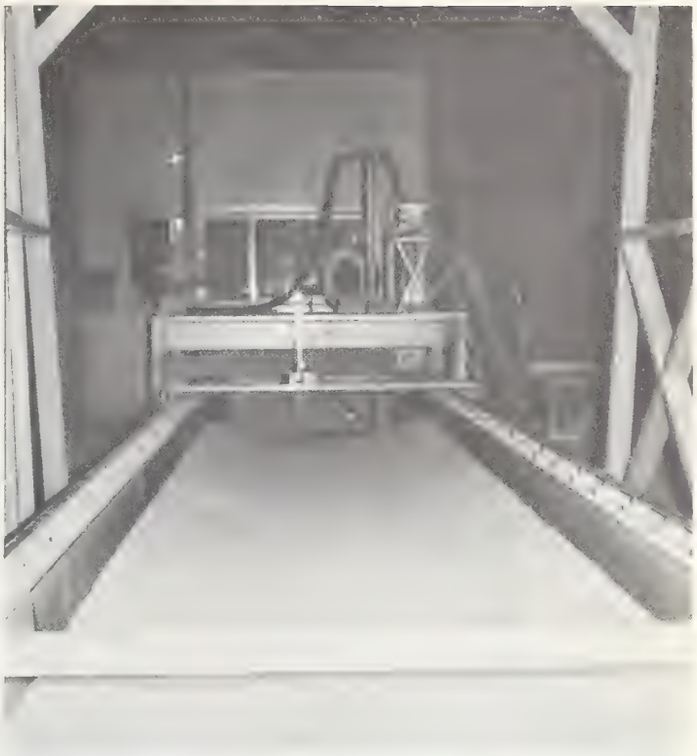


FIGURE 2. — Downstream view of the University of Iowa flume and the sand base for a model channel.

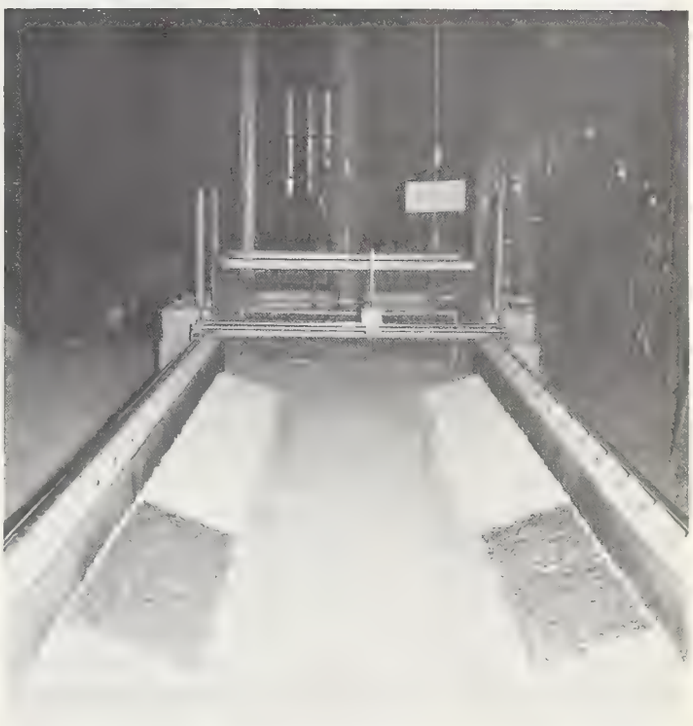


FIGURE 3. — Upstream view of the University of Iowa flume and the rock-lined model channel.

essentially the same degree of boundary roughness as the model riprap.

The material used to construct the channel base and represent the erodible bed was a uniform silica sand that had a median grain diameter of approximately 0.63 mm. A mechanical analysis of the sand is shown on figure 4.

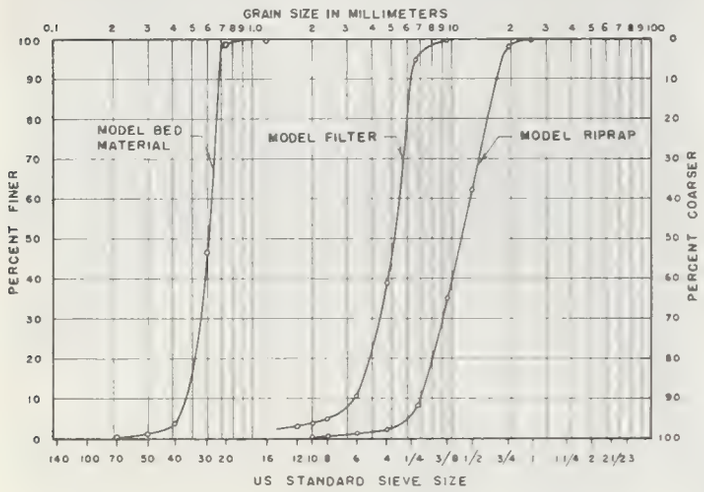


FIGURE 4. — Mechanical analysis of the sand and crushed rock used for model construction.

This sand may not have been the most suitable for the test, due to the excessively large dune formations and their possible effect on the test results. However, it was transported at a rate that gave a reasonable rate of degradation; and sufficient quantities of the sand were readily available in the laboratory.

The filter material and riprap were represented by crushed limestone obtained from a local quarry at Iowa City. Specific gravity of the limestone ranged from 2.713 for the filter to 2.708 for the riprap and derrick stone. Limestone chips with a median grain diameter of 5.25 mm., or 0.207 inches, formed the filter between the sand base and the model riprap. The model riprap was quarry-run crushed limestone that was hand-sieved to remove all material larger than $\frac{3}{4}$ inch in diameter. The resulting median size of the riprap was 0.446 inch, which is equivalent to a prototype diameter of approximately 22 inches, or a weight slightly under 600 pounds. Several other riprap gradations were used in test sections to determine the ability of a smaller rock size to withstand the expected velocities, but for the most part the riprap gradation described above was used throughout the test. Mechanical analyses of the filter and riprap are also shown on figure 4.

Two sizes of model derrick stone were obtained from the same crushed limestone. The larger would pass a 1-inch sieve and be retained on a $\frac{3}{4}$-inch sieve. This represented prototype sizes ranging from about 3 to 4 feet in diameter, or a range in weight from 2,600 to 6,400 pounds. The smaller derrick stone would pass a $\frac{3}{4}$-inch sieve and be retained on a $\frac{1}{2}$-inch sieve. This corresponded to prototype sizes of 2 to 3 feet in diameter, or weights from 800 to 2,600 pounds.

The channel base was formed in the sand with a traveling plywood screed mounted on the flume side rails. A $\frac{3}{8}$-inch layer of filter and slightly over an inch of riprap were placed with the aid of similar screeds. These rock layers were thicker than required by the prototype values, which would have scaled down to $\frac{1}{8}$ inch for the filter and $\frac{1}{2}$ inch for the riprap. The tedious and time consuming hand placement of such thin layers did not seem to be justified.

The sheet piling was represented by a heavy piece of sheet metal, which was either anchored to the flume bottom or in some instances pushed into the sand base and supported only by the surrounding sand. In all cases the top of the sheet metal was at the design bed grade. After placing the rock protection around the sheet metal and along the adjacent side slopes, the bed was filled with sand to the design grade and smoothed so that no part of the structure was exposed at the start of a test series. Before covering with sand, the surface of the rock in selected areas adjacent to the structure was lightly coated with spray enamel. This served as an indicator of damage to the rock protection, since the removal of even a single rock immediately showed as an unpainted area. During construction of the model, as-built cross sections of the channel and structure were measured for use in later analyses. Figures 5 and 6 show photographs of the sill in various stages of construction.

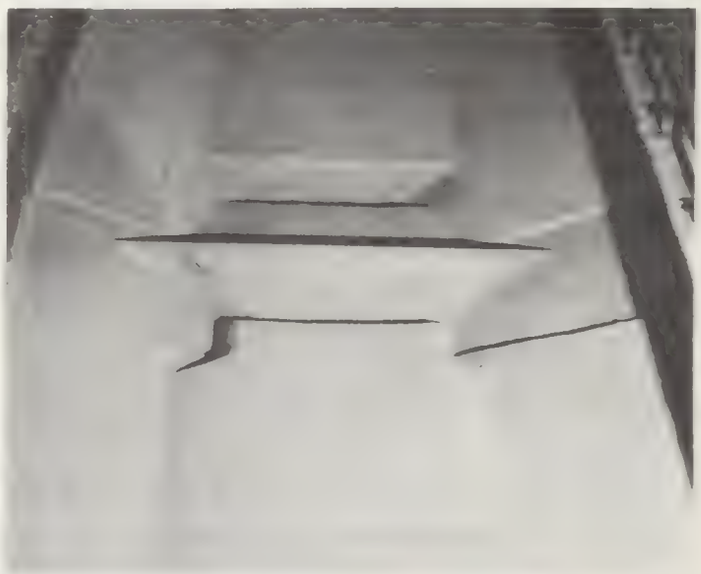


FIGURE 5. — Sand base for the model sill.

FIGURE 6. — Model sill after placement of filter and derrick stone. Flow will be from right to left.

Models Tested

Three separate series of tests were made in order to obtain general design information as well as develop a specific design for the Floyd River. They were as follows:

1. A 1:50 scale three-dimensional study.

2. A two-dimensional study that used the full flume width.

3. A one-half channel study that used one side slope from the three-dimensional study and extended the bed to the opposite wall of the flume.

The half-channel tests and the two-dimensional tests were run in order to extend the information obtained at Berkeley and aid in separating the end effects of the side slopes. Photographs of the two-dimensional, half-channel, and three-dimensional sills are shown on figures 7, 8, and 9.

Since the prototype sills would be spaced too far apart to interfere with each other, only one model still was installed in the flume for a series of tests. The installation of a single sill permitted the flow approaching the sill to become well stabilized and also left a sufficient distance for tailwater recovery downstream of the structure. Several sills in the flume would have interfered with each other to the extent that it would have been almost impossible to obtain the desired information.

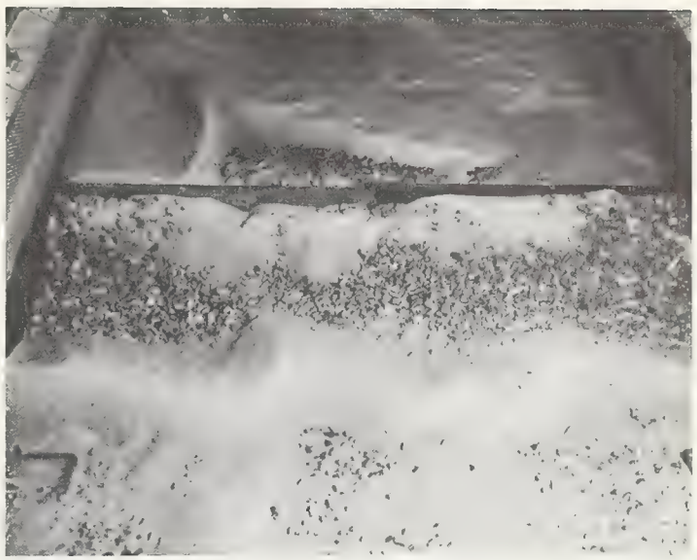


FIGURE 7. — Two-dimensional model sill; flow from top to bottom.

FIGURE 8. — Half-channel model sill; flow from top to bottom.

Preliminary three-dimensional tests were conducted with the V-shaped structure. However, it was soon apparent that this shape was not entirely satisfactory as the V tended to direct the flow against the banks just upstream and cause excessive scour at the point where the sheet piling entered the side slopes. As a result, the design was changed and all subsequent tests were made with a single row of sheet piling at right angles to the channel center line. This alinement minimized the tendency for attack on the side slopes upstream of the structure.

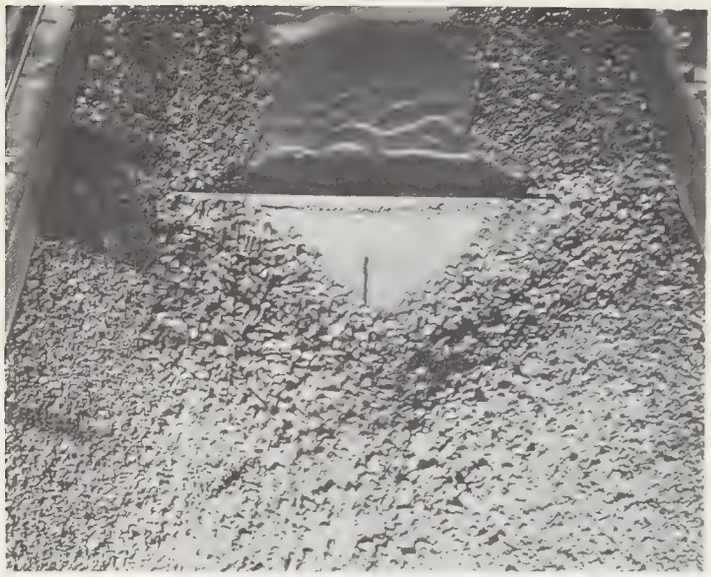


FIGURE 9. — Three-dimensional model sill; flow from top to bottom.

Test Procedure

The test procedure began with slowly back flooding the flume to avoid the damage from water flowing down a dry channel and pouring over the structure. The desired discharge was then established and the tailgate adjusted to give the desired tailwater conditions. If the test was started with the bed at the design grade, the bed was allowed to degrade until the sill was exposed. In any case, the discharge and bed configuration were allowed to become well stabilized before any profile measurements were attempted. After stable conditions were established two water surface profiles were measured with a traveling point gage. The water surface elevations were read at the third points of sections spaced a maximum of 2 feet along the flume and closer in the vicinity of the sill. During certain runs several velocity grids were measured with a pygmy current meter to see if the rough rock boundary of the channel was effective in keeping the high velocity filaments away from the channel side slopes. Visual observations were also made of the flow conditions downstream of the sill for various tailwater stages.

After measuring the water surface profiles, the flow was shut off and the flume allowed to drain. A sufficient number of bed elevations were then measured to determine the average bed profile. Maximum longitudinal spacing was 1 foot. The transverse spacing depended upon the width of the bed but was generally never greater than 1 foot. For use in determining the flow area, cross sections were periodically measured to supplement the bed profile measurements.

Throughout each run the scale recording the amount of sand caught in the trap was read to provide a record of the bed transport. When the sand trap was filled, it was emptied and the sand replaced in the bed and the sand feed elevator. At first an attempt was made to establish for each run an equilibrium bed transport in which the same rate of sand was being fed into the channel as was being trapped at the channel exit. When it became apparent that this would have no significant effect on the energy losses, the bed was allowed to degrade as the tests proceeded. Water surface profiles were thus obtained for varying amounts of sill exposure. Severe degradation downstream of the structure was permitted in order to determine the stability of the sill with large amounts of downstream erosion and also the ability of the sill to stabilize the upstream bed grade.

Development of the Rock Protection

Determination of the amount of rock protection required for the sheet piling was carried on concurrently with the measurement of head losses over the structure. At first the upstream riprap blanket was placed 5 feet below the top of the sheet piling and extended horizontally upstream for a distance of 50 feet. After several runs, profiles over the sill showed that rock from the surface of the blanket was being carried away. The remaining rock assumed a stable slope of 1 on 5 to 1 on 6 in the upstream direction. The upstream blanket was then revised to slope in the upstream direction at 1 on 6 with the horizontal. It was also found that derrick stone was required in this blanket for the first 12 feet upstream of the sheet piling. Derrick stone was initially placed on the entire side slope from 50 feet upstream to 50 feet downstream of the sheet piling.

As testing progressed, it was found that very little derrick stone was required on the side slopes above the sill crest or upstream of the piling, but additional derrick stone was required further downstream. From 12 feet upstream of the sheet piling to 50 feet downstream of the piling, derrick stone was required below an elevation 5 feet above the sill crest. From 50 feet downstream to the end of the derrick stone 100 feet downstream of the piling, the top of the derrick stone was sloped from 5 feet above the crest to 5 feet below the crest. The bottom of the derrick stone on the downstream side slopes was carried to 16-feet below the design bed. On the 1-on-2 apron downstream of the sill the original rock riprap was found to be satisfactory, except for a narrow strip along the intersection with the side slope. Here was found to be the area of greatest stress on the rock protection, and the side slope derrick stone was extended onto the downstream apron in this area. A plan view of

the final design of the sill is shown on figure 10, and sectional views are shown on figure 11.

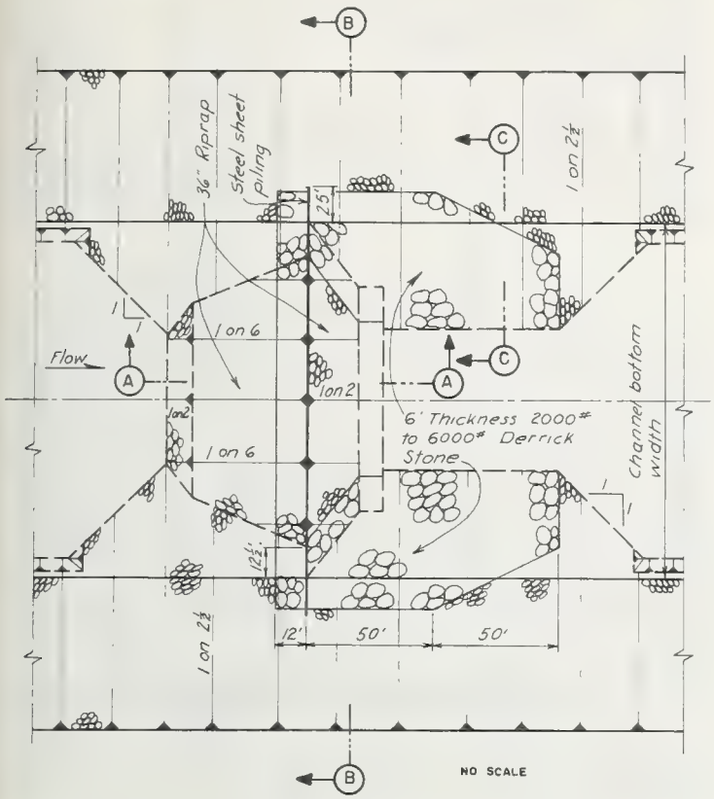


FIGURE 10. — Plan view of the final sill design.

Flow Condition Created by the Structure

The flow conditions in the vicinity of the sill are to a great extent dependent upon the tailwater stage. Upstream of the sheet piling the surface of the rock blanket was generally swept clean of sand and exposed for all tailwater stages. This was due to the turbulence caused by the discontinuity in the bed created by the projecting sheet piling. Downstream of the structure the flow pattern ranged from only minor surface disturbances to a well-developed hydraulic jump. Observations during the tests indicated that for ratios of the tailwater depth to the critical depth, T'/D_c, greater than approximately 1.65, the disturbance was minor with only a few ripples and undulations in the water surface. For lower tailwater stages where T'/D_c lay between 1.65 and 1.45, large standing waves and undulations developed downstream of the structure for a distance of several hundred feet. When the ratio of T'/D_c fell to about 1.45, a definite surface roller developed which became a well-established hydraulic jump for lower tailwater stages.

If the tailwater depth did not drop much below critical, the high velocity jet tended to float on the surface and did not attack the downstream bed. A reverse roller existed under the high velocity jet and actually deposited material back against the downstream face of the structure. Figure 9 shows this deposit of stand remaining while all the rest of the bed material

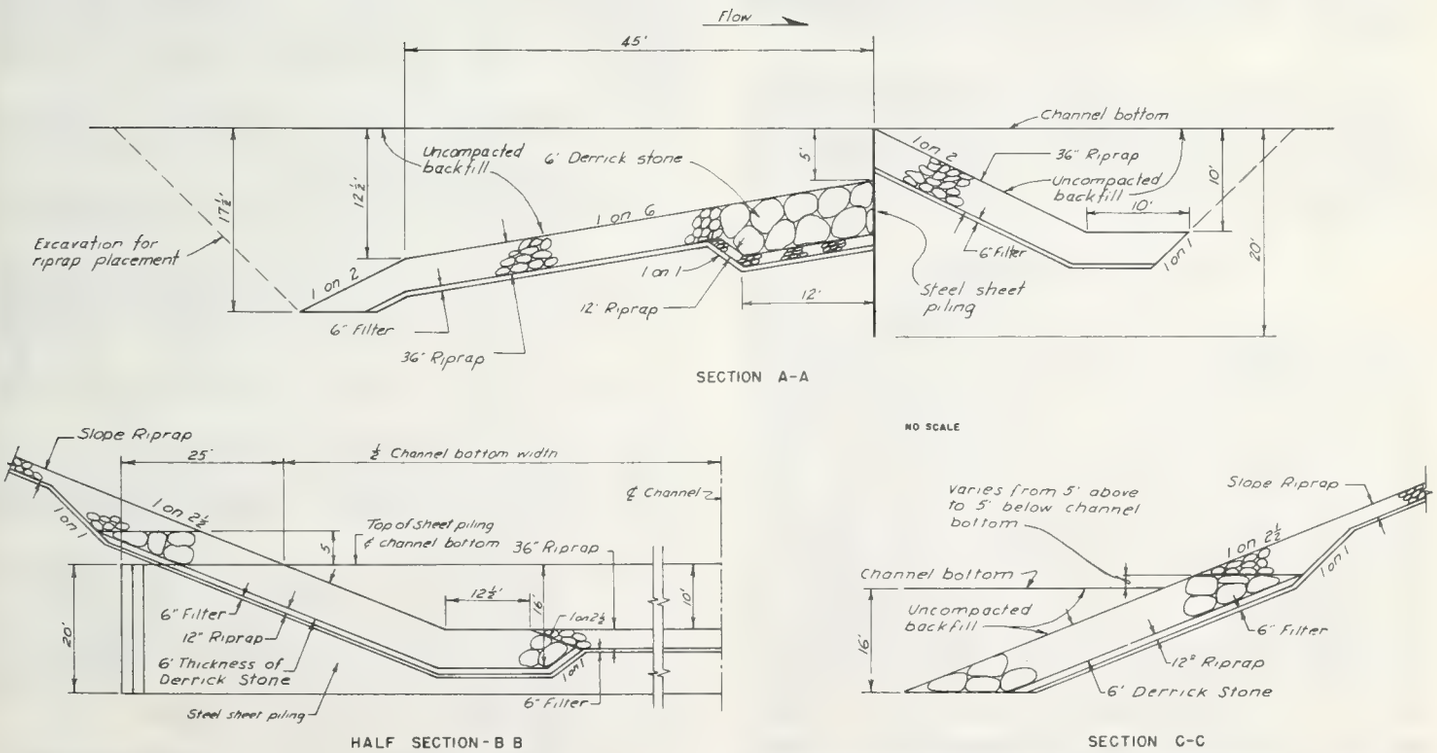


FIGURE 11. — Sectional views of the final sill design.

farther downstream has been carried away. If the tailwater dropped significantly below critical, the plunging nappe rapidly scoured a hole that undermined the rock on the downstream apron.

The turbulence near the water surface downstream of the structure tends to be confined to the central part of the channel by the lateral flow at each end of the crest. This lateral flow develops from the narrower water surface width downstream of the structure created by the change in water surface elevations in the trapezoidal channel. Figure 12 shows the downstream disturbance resulting from the design discharge and nearly 5 feet of drop in water surface over the structure; the turbulence is confined to the center of the channel and well away from the side slopes.

FIGURE 12. — Downstream surface turbulence resulting from the design discharge over the three-dimensional model.

Analysis of Data

The average water surface elevation and bed elevation measured at each cross section were plotted to obtain water surface and bed profiles. Upstream of the sill the best straight lines through the plotted bed and water surface profiles were determined by the method of least squares. The upstream energy grade line was then computed, based on the straight line bed and water surface. This energy grade line was then projected to the sill crest. Downstream of the structure it was generally impossible to use the least squares method to make a straight line determination of the bed or water surface due to the recovery zone of the water surface and the scour hole that developed in the bed. Instead the energy gradient was computed on the actual measured bed and water surface at each cross section. It was then usually possible to fit a straight line to the energy grade line computed in this manner. This downstream energy grade line was then projected to the sill crest. The flow area in a section was usually obtained by assuming a trapezoidal channel with the as-built side slopes. If it appeared the cross sections deviated significantly from this assumed geometry, the flow area was based on representative cross sections actually measured after each run. Figure 13 shows typical plotted profiles of the bed and water surface and the lines of best fit for the profiles.

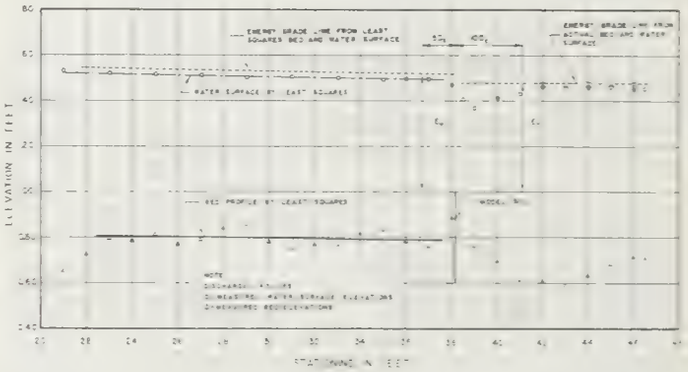


FIGURE 13. — Typical bed and water surface profiles from the University of Iowa tests.

The plotted profiles and the additional data collected during the testing gave the information required for analysis of the data. A brief definition of the symbols used in the following discussions and on the design curves is given below.

E_H = Headwater energy above the crest elevation based on a projection of the upstream energy grade line to the crest.

E_T = Tailwater energy above the crest elevation based on a projection of the downstream energy line to the crest.

E'_H = Headwater energy above the crest elevation at 5 critical depths upstream of the sheet piling.

E'_T = Tailwater energy above the crest elevation at 10 critical depths downstream of the sheet piling.

D_C = The critical depth in the cross section at the sill crest.

E_C = Critical energy in the cross section at the sill crest.

H' = Headwater depth of flow above the crest elevation at 5 critical depths upstream of the sheet piling.

T' = Tailwater depth of flow above the crest elevation of 10 critical depths downstream of the sheet piling.

P = Projection of the sill above the average upstream bed as determined by projecting the straight line representing the bed profile downstream to the sheet piling.

Q = Total discharge in the channel.

Q_v = Apparent discharge in the triangular-shaped end sections of the trapezoidal and half-channel.

$Q_{\sqcup}$ = Apparent discharge in the central portion of the trapezoidal and half-channel.
b = The crest width.
A = The total flow area between the crest elevation and the water surface elevation 5 critical depths upstream of the sheet piling.
A_v = The area in the triangular-shaped end sections between the crest elevation and the water surface elevation 5 critical depths upstream of the piling.
C = Coefficient of discharge for the total flow area.
C_v = Coefficient of discharge for the triangular end sections.
g = The gravitational constant
D_{15} = The grain diameter at which 15 percent, by weight, of the material is finer.
D_{50} = The median grain diameter.
D_{85} = The grain diameter at which 85 percent, by weight, of the material is finer.

The first attempt at analysis of the data was to make a dimensionless plot of E_H/D_C versus E_T/D_C for all the profile data. Examination of the plot immediately indicated separate curves for the two-dimensional channel data and for each discharge in the trapezoidal and half-channel. It was therefore necessary to devise some means of separating the effects of the triangular end sections from the essentially two-dimensional flow in the central part of the channel. This was accomplished by plotting the two-dimensional data in the dimensionless form of E_C/E'_H versus E'_T/E'_H. These points seemed to define a single curve, shown on figure 14, which is assumed to be applicable for wide rectangular crests. The apparent flow through the triangular end sections in the trapezoidal and half-channel tests was then computed by assuming the curve on figure 14 defines $Q_{\sqcup}$, the flow over the horizontal part of the crest. From the value of E'_T/E'_H measured during the test, the value of E_C/E'_H was determined from figure 14. E_C was then computed, based on the value of E'_H measured during the test. The corresponding D_C for a horizontal crest was then computed from $D_C = 2/3\ E_C$, and the apparent discharge over the horizontal crest was computed from $Q_{\sqcup} = bq = b\sqrt{gD_C^3}$. By subtraction the apparent discharge through the end sections became $Q_v = Q - Q_{\sqcup}$. An orifice-type coefficient for the flow in the end sections was computed from $C_v = Q_v/A_v\sqrt{2g}\ (E'_H - E'_T)$. The values of C_v thus obtained were plotted against E'_T/E'_H as shown on figure 15. Although there is considerable scatter, a relationship for estimating the flow through the end sections seems to be defined in figure 15, and it could be used in conjunction with figure 14 to compute head-discharge relation for the sills in this study.

The analysis just discussed, however, leaves something to be desired, considering the uncertainty suggested by the scatter in figure 15.

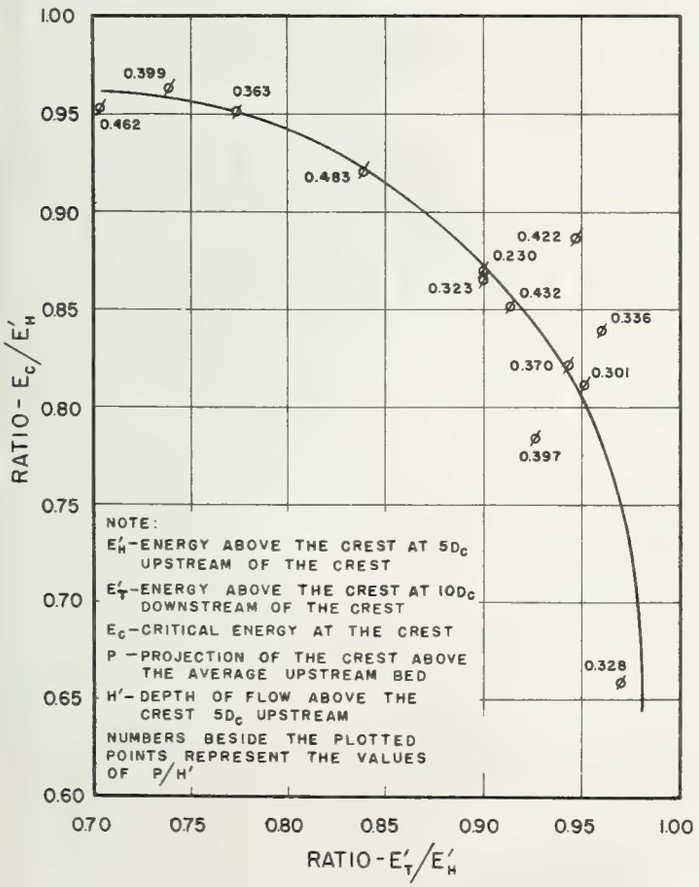


FIGURE 14. — Energy losses for a two-dimensional sill.

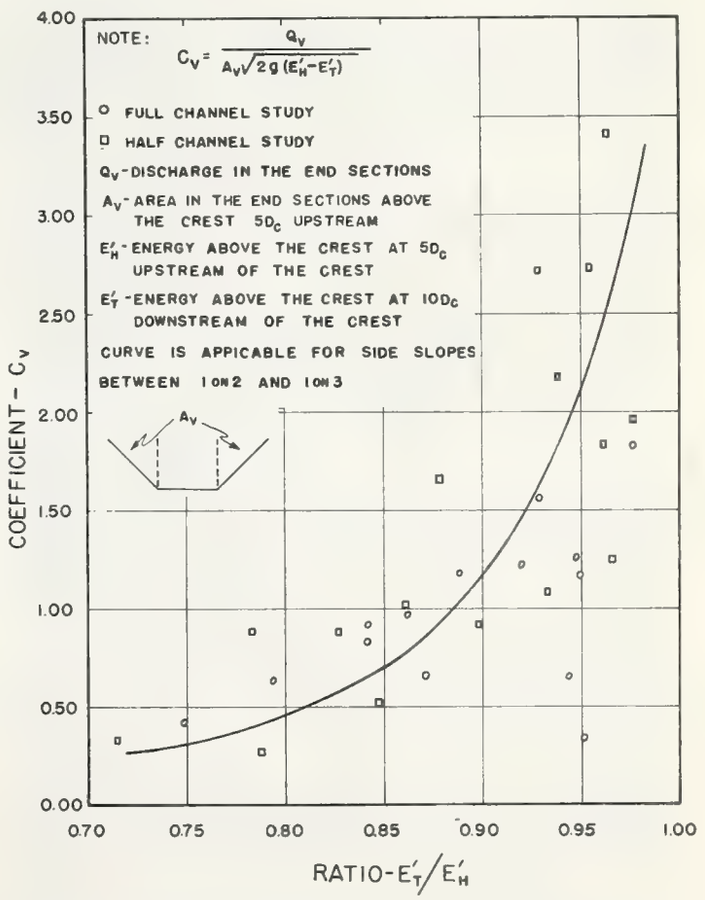


FIGURE 15. — Discharge coefficient for the end sections of a sill in a trapezoidal channel.

This method also involves a trial and error computation to determine the energy loss over a sill, if the normally known values of discharge and tailwater stage are given. Therefore, further analysis of the data was made to see if a more consistent relationship might be found and the computation for the upstream water surface simplified. Figure 16 shows a relationship between a total discharge coefficient defined from $C = Q/A\sqrt{2g\,E'_H}$ and the dimensionless submergence ratio T'/D_C, where D_C is the critical depth in the trapezoidal cross section at the sill crest. The number beside each plotted point represents the ratio of the area in the end sections to the total area. Two enveloping curves have been drawn through the plotted points. The upper curve is for the case of a rectangular channel where A_v/A equals zero. The lower is the estimate curve for A_v/A equals 0.30, which is the case where the area in the end sections is approximately one-third the total flow area. Of many possible combinations of data, this was the only one in which the two-dimensional data defined a boundary or limit and the data from the half-channel and trapezoidal channel fell in a position with respect to the relative amount of the area in the end sections. If the tailwater stage and discharge are known and an estimate can be made of the value of A_v/A, the upstream energy can be computed once the corresponding value of C is found from the plot. Figure 16 has been adopted for design purposes.

The scatter and lack of clear definition in figures 14 and 15 may be due to a number of reasons. One is the error in measuring the water surface. For high discharges and high degrees of submergence, the water surface was sufficiently rough to make the error in determining the water surface approach the magnitude of the losses being measured. The convention used in evaluating the discharge in the end sections leaves much to be desired when one assumes that the center part of the sill behaves as a two-dimensional sill with no influence from the end sections. The vertical walls at the edges of the channel may have had some influence in creating higher upstream stages over what would have been experienced with a true trapezoidal channel. The relatively large dune formations on the movable bed upstream of the sill undoubtedly influenced the tests results to some extent, as the dune height approached 20

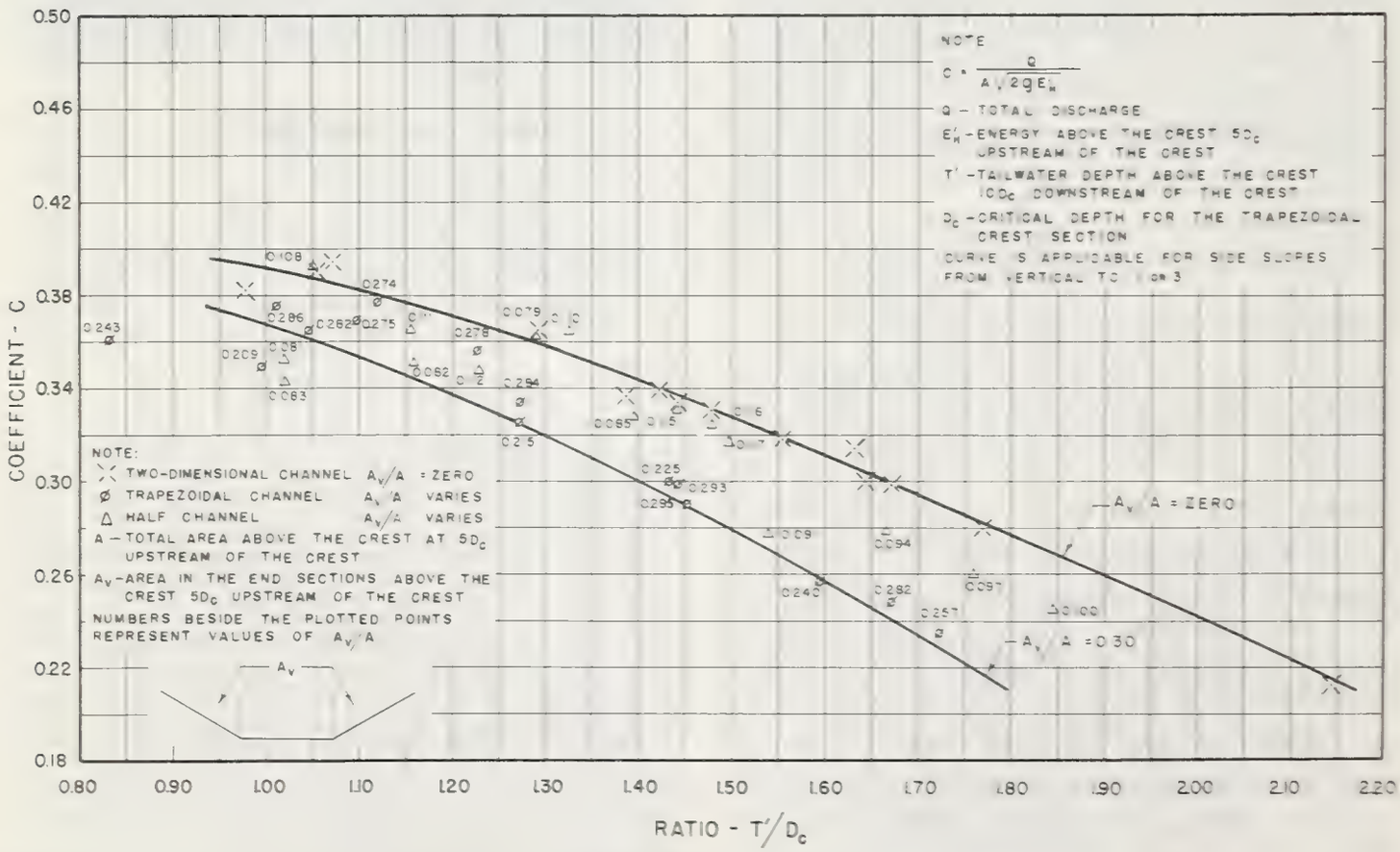


FIGURE 16. — Total discharge coefficient for a sheet piling and rock sill.

to 25 percent of the water depth in many cases. The effect of a dune temporarily burying the sill should not be reflected in the data, as water surface profiles were generally not measured while this condition was observed.

No definite relation between the degree of exposure of the structure above the bed and the amount of loss over the sill is indicated by the data obtained in the tests. On figure 14, where the value of P/H' is shown beside each plotted point, there is no apparent correlation between the values of P/H' and their location with respect to the curve. This may be due to the nearly constant degree of local exposure of the piling. Unless the sill is temporarily buried by a passing dune, the local turbulence created by the projecting piling will keep the upstream rock blanket swept clean of material. Even though the general bed elevation of the upstream channel may vary considerably from the elevation of the surface of the blanket, it will form a streamlined approach to the surface of the rock. The approach losses become only a small part of the total energy loss, since most of the loss is created by the local turbulence at the piling. For any given discharge, the loss due to this turbulence will remain nearly constant as the rock blanket places a limit on the amount of piling exposure that can take place.

Application of the Structure and Suggested Design Criteria

The application of the structure is limited in that it must essentially operate as a submerged sill where the high velocity jet will tend to float on the surface rather than plunge to the bottom. This requirement can be met if the tailwater does fall below approximately 0.8 of the critical depth at the crest section. If the tailwater is lower, the nappe will plunge and rapidly destroy the structure either by direct attack on the downstream slope or by creating a scour hole that will undermine the structure. The only time this tailwater criteria may be violated is for relatively low base flows, in which case adequate rock sizes on the downstream slope must be provided. If the above tailwater criteria can be met, the structure has many possible applications where it is desired to control water surface elevations and retard head cutting. The structure will not act as a barrier to bed transport except under low flow conditions, and it will not directly protect the upstream channel from scour. Erosion upstream of the structure will be controlled primarily through reduced velocities created by the increase in water surface elevations induced by the sill as it becomes exposed. Controlling the erodible bed of a high velocity channel and controlling bed and bank erosion that threatens damage to bridges or structures adjacent to the channel are examples of situations where the sill would be suitable. A decided advantage of the structure is the flexibility of the rock to adjust to conditions that would destroy a conventional concrete structure. If tailwater conditions prevent the use of a single sill, it may be possible to meet the tailwater criteria and raise the water surface to the desired elevation by the use of a series of sills.

Design of the structure starts with a determination of the tailwater rating curve and then proceeds with the selection of a crest elevation that satisfies the condition that T'/D_C must be greater than 0.80. The crest width should be essentially the same as the width of the upstream and downstream channel in order to avoid the inherent difficulties with a constriction where damaging lateral eddies will develop downstream. The headwater rating curve may be determined either with the curves on figures 14 and 15 or with the curves on figure 16. The curves on figures 14 and 15 should not be used for side slopes steeper than 1 on 2 or flatter than 1 on 3. If a reasonable estimate of the ratio A_v/A can be made, the curves on figure 16 will be simpler to use and can probably be applied to any side slope from vertical to about 1 on 3. If the headwater rating curve indicates that a single structure does not create the required head losses, it will be necessary to add additional sills until the desired water surface elevation is attained.

The type of sheet piling and the depth to which it must be driven will depend more upon construction conditions than hydraulic requirements. Since one or both sides of the piling will be exposed to considerable depth during construction, the piling should be designed to withstand the load created by a saturated fill on one side and exposure to the depth required by construction on the other side. It should also be capable of withstanding the impact of large rocks being placed against it. The sheet piling should extend into the side slopes far enough so that it will not be outflanked if a considerable amount of rock loss or adjustment takes place at the ends of the sill.

If the sill is intended for use in a channel where no bank revetment is contemplated, the minimum amount of channel side slope protection should extend from the upper end of the upstream blanket to the end of the derrick stone downstream of the sheet piling. Upstream of the piling the side slope protection should tie into the top upstream blanket. Downstream of the piling it should tie into the top of the derrick stone. It is suggested that the determination of the required rock size for the side slopes be

based upon the following relationship developed by Isbash.[1]

$$V = Y\sqrt{\frac{2g\ (S-W)}{w}}\ \sqrt{D}\ \sqrt{\text{Cos}\ \alpha}$$

Where V = velocity against the stone,
Y = numerical constant with a range in value from 0.86 to 1.20,
S = specific weight of the rock,
W = specific weight of water,
D = diameter in feet of a sphere of equivalent weight,
α = the angle between the horizontal and the bed upon which the rock lies.

For the rock on the channel side slopes, a value of 1.20 for Y would be appropriate, since the velocity distribution in the channel is well developed. For more severe conditions when the flow impinges directly against the rock, the more conservative value of 0.86 for Y should be used. The value for Cos α should be taken from the angle between the side slopes and the horizontal.

The surface of the upstream blanket should be from 3 to 5 feet below the top of the sheet piling and should be placed on a 1-on-5 to 1-on-6 slope in the upstream direction. It should extend upstream from 1½ to 2 times the expected head on the structure and should terminate in some form of toe to prevent raveling and undermining of the upstream edge. Except for the area immediately upstream of the sheet piling, the same rock size gradation as used on the channel side slopes will be adequate.

The downstream apron of the structure requires rock protection from the top of the sheet piling to ¾ to 1 critical depth below the top of the piling. This rock should be placed on a slope no flatter than 1 on 2, since flatter slopes may become subject to attack from the high velocity nappe. A short horizontal apron at the bottom of the 1-on-2 slope may be desirable to aid in turning high velocity filaments and directing them parallel to the bed. Except for the corners where the downstream apron and side slopes intersect, extra heavy rock protection is not required on the downstream apron. However, the rock should be of sufficient size to withstand the low flows that may plunge onto it. The previously discussed relationship developed by Ishbash should be used for determining the required rock size in this area. Since the low flows may plunge directly onto the rock, a value of 0.86 for Y would be appropriate. This value of Y will give essentially the same rock size as suggested for similar conditions by the Bureau of Reclamation in their Hydraulic Laboratory Report Hyd–409.[2]

Three areas in the vicinity of the structure are subject to above normal stress from the flow conditions created by the sill, and heavy rock protection is required in the form of derrick stone. These areas are (1) just upstream of the sheet piling, (2) in the corners where the downstream apron and channel side slopes intersect, (3) on the lower side slopes downstream of the crest. The plot on figure 17, which was developed from observations made during the study, shows a relationship between the ratio of tailwater to headwater, T'/H', and the required median diameter of derrick stone. The plotted points for the three rock sizes represent conditions varying from loss of a small amount of rock to essentially complete failure where significant loss of rock and severe slumping of the side slopes at the end of the crest took place. The lower solid curve represents the minimum derrick stone size that should be considered, and the upper dashed curve is the one suggested for design purposes.

The derrick stone on the upstream side of the structure should extend upstream of the piling for a distance of two to three times the vertical exposure of the sheet piling. It should also extend up the adjacent side slopes to about one-third critical depth above the sill crest. Since the stress in this area is not the most severe, the median derrick stone size may be determined from the lower of the two curves on figure 17.

The corners downstream of the sheet piling are subject to severe stress, and it is suggested that the size of derrick stone for this area be determined by the upper curve on figure 17. This heavy derrick stone should extend onto the downstream apron for a distance of 10 to 15 feet. On the lower side slopes it should be carried downstream three to four critical depths from the sheet piling. The lower side slope derrick stone should start about one-third critical depth above the sill crest and should be extended downward to an intersection with the downstream apron. If it is expected that the structure will be subjected to flows of relatively long duration or frequent high discharges with relatively low tailwater stages, it may be desirable to extend the derrick stone to six to eight critical depths downstream of the sheet piling. The rock size for this additional protection could be determined from the lower curve on figure 17.

Figure 10 shows the area in which derrick stone was required for the Floyd River sills. In order to reduce the number of rock sizes re-

[1] ISBASH, S. V. CONSTRUCTION OF DAMS BY DUMPING STONES INTO FLOWING WATER. Sci. Res. Inst. Hydrotech. Leningrad. 1932. Trans. by A. Dovjikov, U.S. Army Corps of Engineers, Eastport District, September 1935.

[2] U.S. DEPARTMENT OF THE INTERIOR. STILLING BASIN PERFORMANCE STUDIES, AN AID IN DETERMINING RIPRAP SIZES. Hydraulic Lab. Rpt. Hyd-409, U.S. Bur. of Reclam. February 1956.

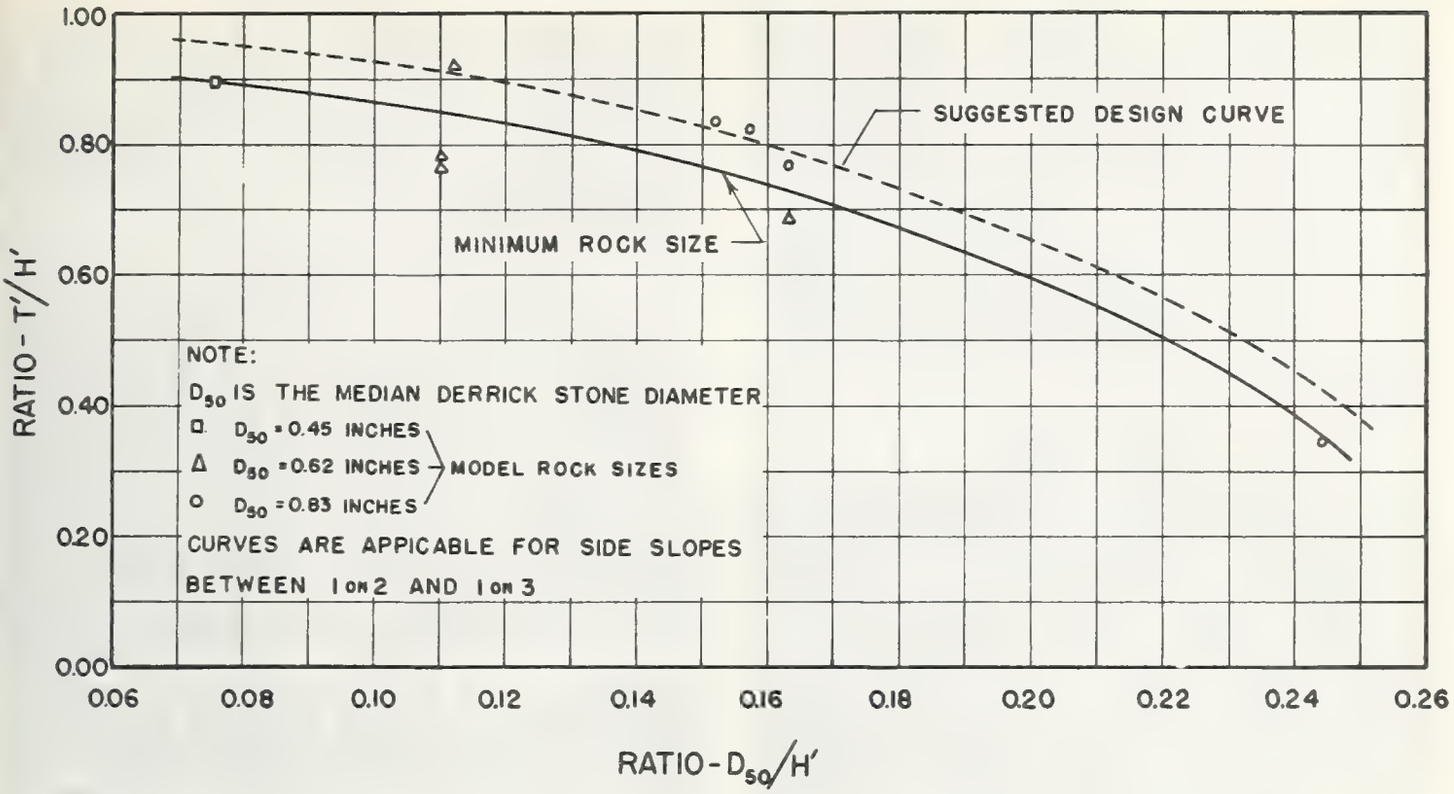


FIGURE 17. Derrick stone sizes required for sill protection.

quired, the same gradation of derrick stone was used throughout the structure. If it should prove economical to specify several gradations of derrick stone, the area may be zoned in the manner illustrated in the preceding paragraphs.

The recommended layer thickness for all of the rock in the vicinity of the structure is two median rock diameters, and in no case should it be less than 1½ times the median rock diameter. It is also very important that an adequate filter be placed between the rock and the bed material. The corners downstream of the piling are subject to severe piping of bed material through the voids in the surface rock. In runs with low tailwater stages, the limestone chips used for the filter material could often be observed to literally stream through the voids in the model derrick stone. This is indication of the very strong lift forces that exist in this area. It is recommended that the relation developed by Terzaghi in conjunction with the Corps of Engineers [3] be used for the design of the filter. This relationship is as follows:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Base}} < 5$$

$$4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Base}} < 20$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 25$$

If the surface rock is derrick stone, several layers of filter material may be required and the above conditions must hold for successive layers if an adequate filter is to be provided.

Areas for Additional Investigation

Although the study developed an adequate design for the Floyd River, additional investigation in several areas would be desirable. Additional information is needed on the minimum rock size required in the various areas of the rock protection for the sill. Stability of the rock is a function of factors other than the velocity of the flow. For example, in the corners downstream of the crest damages to the rock blanket often resulted from the individual rocks being bodily picked off the surface and carried away by the flow rather than being merely rolled down the slope.

Another interesting aspect is the effect of

[3] POSEY, C. J. SOME BASIC REQUIREMENTS FOR PROTECTION AGAINST EROSION. Minn. Internatl. Hydraul. Convention Proc. 1953, pp. 85-87.

TERZAGHI, KARL, and PECK, RALPH B. SOIL MECHANICS IN ENGINEERING PRACTICE. John Wiley and Sons, Inc., New York. 1948.

U.S. WATERWAYS EXPERIMENT STATION. INVESTIGATIONS OF FILTER REQUIREMENTS FOR UNDERDRAINS, FIELD AND LABORATORY INVESTIGATION OF DESIGN CRITERIA FOR DRAINAGE WELLS, and LABORATORY INVESTIGATION OF FILTERS FOR ENID AND GRANADA DAMS. U.S. Waterways Expt. Sta. Tech. Memo. 183-1, 195-1, and 3-245, respectively.

sediment transport on the flow regime over the structure. Whether this is due to streamlining of the structure or a change in fluid properties cannot be determined by the results of these tests. In the Berkeley tests the extremely heavy bedload seemed to reduce the energy losses compared to a sill with no bed transport. The tests at Iowa City generally did not have a heavy rate of bed transport. However, it was observed that when a dune was passing over the structure, the turbulence downstream of the sill was reduced and the headwater seemed to lower slightly.

Conclusions

The study successfully demonstrated the ability of the structure to perform as desired, and approval for construction of the high velocity channel was granted by the Iowa Natural Resources Council. Computations based on the test results showed that five sills spaced approximately 2,000 feet apart in the lower 1½ miles of the Floyd River would effectively maintain the water levels in the channel despite considerable bed scour. Computations showed that even if the bed was to degrade 10 feet, the water surface would be maintained within 1 or 2 feet of the water surface for no degradation. The structure can provide an economical solution to problems of a similar nature where it is desired to control bed degradation and the water surface elevations, provided the tailwater conditions will allow the sill to essentially operate as a submerged structure.

Acknowledgments

The author wishes to express his appreciation to Hunter Rouse and Lucien Brush of the Iowa Institute for Hydraulic Research. Without their cooperation and advice, the laboratory testing could not have been successfully completed. Alfred S. Harrison, Chief of the Hydraulics Section, Missouri River Division, gave much helpful advice in the preparation of this paper. Warren Mellema, Omaha District, assisted with much of the laboratory work and the analysis of the data; Wayne Stufft, Omaha District, made the initial studies at the University of California. The writer also wishes to express his thanks to John Bath, Lewis Hulman, Gary Staley, D. William Saint, and Frank Vovk, all of the Omaha District, for their assistance in the laboratory.

COMPUTING SUSPENDED SAND LOADS FROM FIELD MEASUREMENTS

[Paper No. 56]

By Alfred S. Harrison and Harold C. Lidicker, *hydraulic engineers, U.S. Army Engineer Division, Omaha*

Synopsis

On the Missouri River the bulk of the bedload material is transported in suspension with low concentrations near the water surface and high concentrations near the bed. Because of this nonuniform distribution of particles in suspension, a large part of the sands coarser than 0.10 mm. is carried within a half foot of the bed below the lowest level that can be sampled. It is necessary, therefore, to determine through field measurements the vertical distributions of velocity and concentration in the part of the flow that can be measured, and then to extrapolate these distributions toward the bed in order to compute total suspended sand load. The most reliable procedure is direct measurement of these distributions by obtaining a series of point velocities and point samples in the vertical. Velocities are relatively economical to obtain, as they involve only some extra field work. The samples, however, are relatively expensive, as they involve costly laboratory analysis, not only for concentration but also for mechanical analysis, since it is desired to compute the loads for individual grain-size fractions.

In the interest of economy a procedure has been devised that requires only two sediment samples in each vertical, one depth-integrated sample and one point sample at a reference depth, obtained with a P–46 sampler. A vertical velocity distribution is also measured with a current meter. From these field data the concentration distributions in the measurable part of the flow are deduced by a method developed from relationships presented by Vanoni and Einstein. This method gives a more reliable determination of suspended load than the modified Einstein method of the U.S. Geological Survey, because it uses a direct measurement of the velocity distribution and requires fewer assumptions in deducing the sediment concentration distributions.

Criteria for how close to the bed to extrapolate the measured velocity distribution and the deduced sediment distributions have also been developed. The integration of the velocity and sediment distributions to obtain total suspended load follows the method of Einstein. In the Missouri River, no attempt is made to estimate bedload, as this is small compared with the suspended load.

The Problem

The determination of bed material loads in the Missouri River has become particularly important since Gavins Point Dam was closed in 1955 and now that the navigation and bank stabilization project below Sioux City is nearing completion. The aim of the Corps of Engineers sedimentation program on the Missouri River is continuous evaluation of the changes in river regime brought about by these projects in order to foresee any adverse effects in time for remedial action, to provide data needed in the design of future work, and to contribute to the general knowledge in the field of river engineering. The changes in the riverbed, its elevation, its grain-size composition, and its roughness configuration are of particular significance in this evaluation — and these changes are directly related to the bed material load, defined as the transport rate of those grain sizes that are also found in significant quantities in the bed.

The riverbed material for over a hundred miles below Gavins Point Dam is a fine sand with median grain diameter 0.2 mm. Very little is finer than about 0.06 mm. or coarser than about 0.5 mm. Occasional gravel pockets are found, usually at the downstream toes of sandbars. The river slope is about 0.9 foot per mile. Water depths normally range up to 25 feet in a cross section, with a mean depth around 10 feet, and velocities average 3 to 4 feet per second. Under these conditions the bed material load is transported mainly in suspension. Transport as bedload is a very small part of the total, although for the coarsest grain-size fractions bedload may be more significant.

Two factors complicate the determination of suspended sand loads from field measurements. Since neither the velocities nor the concentrations are uniformly distributed in the vertical, a number of points must be measured in order to define the vertical distributions. This is not only time-consuming in the field but costly in the laboratory. Each sample must be analyzed, not only for concentration but for mechanical analysis, because sand loads are desired for each individual grain-size fraction. Figure 1 shows typical vertical distributions of velocity, obtained with a Price current meter, and concentration, obtained from P–46 samples, for the Missouri River at Omaha.

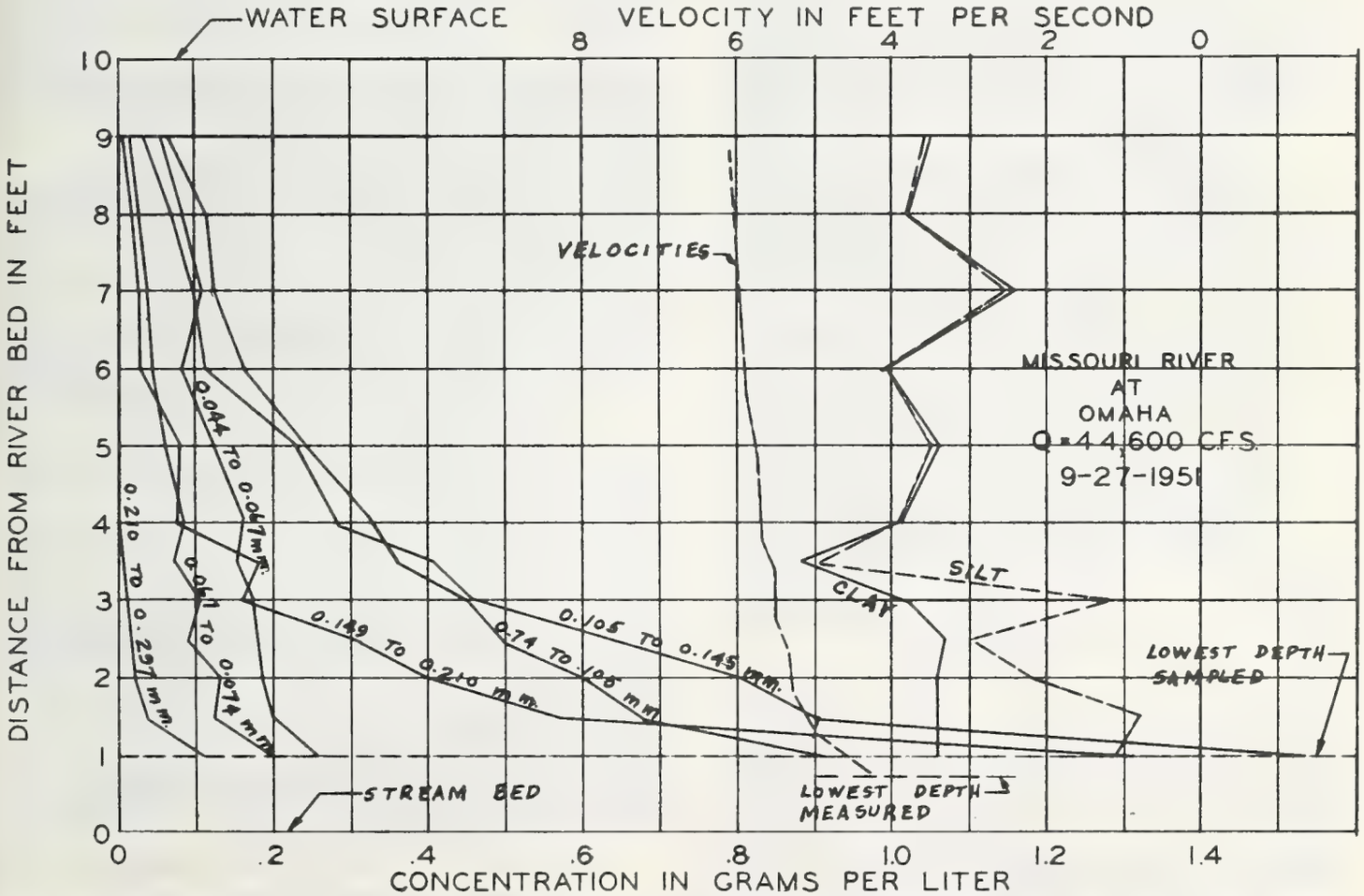


FIGURE 1. — Vertical velocity and concentration distributions in Missouri River.

The second complicating factor is that a significant part of the sand load is carried below the lowest point that can be sampled with the suspended load sampler. This is due to the relatively high concentrations near the bed as compared with the water surface. The nearest approach to the bed that is practical with the Price current meter or with the P–46, the D–43, or the D–49 samplers is about 0.5 foot. It is, therefore, necessary to extrapolate the measured distributions toward the bed in order to determine the total sand load. Figure 2 shows

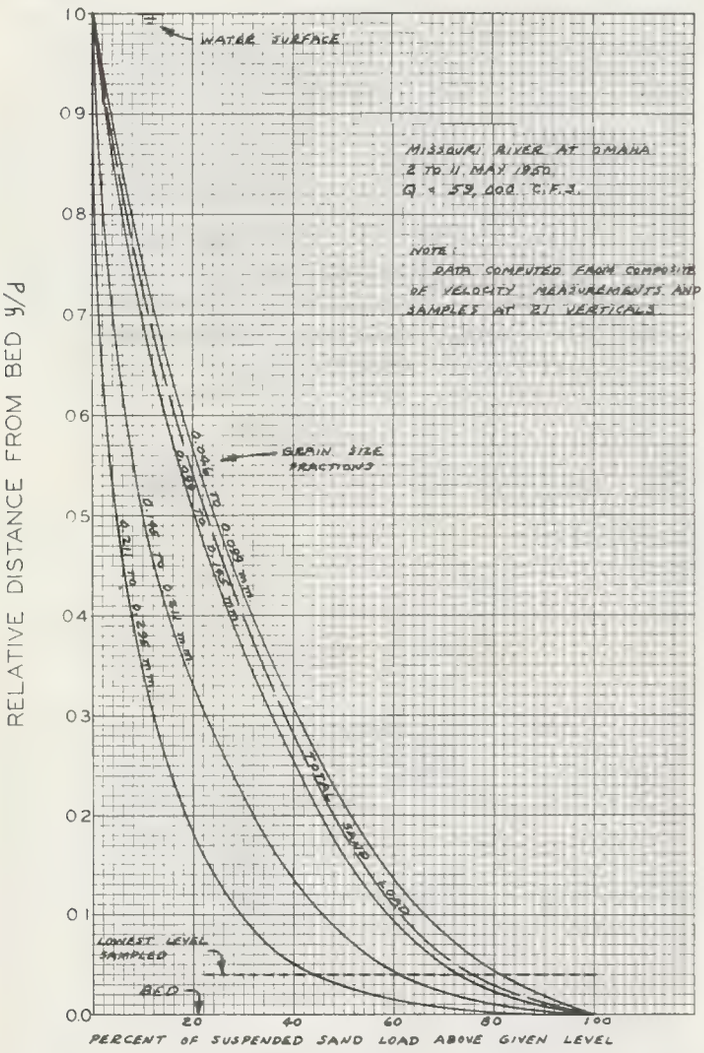


FIGURE 2. — Vertical distribution of suspended sand load in Missouri River.

typical distributions of sand loads in the vertical by grain-size fractions for the Missouri River at Omaha. These curves were computed below the lowest measured points by the extrapolation method to be described later in this paper. A distance from the bed equal to about 0.04 of the depth is indicated in figure 2 as the lowest level that could be measured. The curves in figure 2 show that 75 percent of the total sand load is carried above this level. For the finer grain-size fractions, this includes the bulk of the load. For the coarser sand fractions, however, a smaller part of the load is measured directly. For example, only 45 percent of the load in the 0.211- to 0.295-mm. range is carried within the measured zone; and the rest of the load in this size range must be based on extrapolation of measured points. Since the median grain size of the material in the riverbed is about 0.2 mm., the transport rates for the coarser suspended sand fractions are important in evaluating changes in the riverbed regime.

Objectives

The analysis that follows will lead to four objectives: (1) to obtain relationships based on the measured data that permit extrapolations of velocity and concentration distributions downward below the lowest level of measurement; (2) to establish tentative criteria for the lower limit of integration — or in other words, how close to the bed to extend the velocity and concentration distributions; (3) to develop analytical procedures to facilitate the integration of concentration times velocity over the water depth to obtain sediment load; and (4) to develop a procedure requiring the minimum amount of field measurement and laboratory analysis to define the velocity and concentration distributions in the measurable part of the flow.

Nomenclature

A = a/d, a dimensionless expression for a.
a = The height from the bed of the lower limit to which the suspended load integration is carried (in feet).
α = The height from the bed of the lowest level sampled by a depth-integrating sampler (in feet).
C_α = The average concentration for a given grain-size fraction from the water surface down to a distance α from the bed, as measured with a depth-integrating sampler (grams per liter).
C_p = Known concentration of a given grain-size fraction at a distance p from the bed (grams per liter).
C_{pt} = Known concentration of all sand grain-size fractions at a distance p from the bed (grams per liter).
C_y = The concentration of a given grain-size fraction at a distance y from the bed.
C_{yt} = Total concentration of all sand grain-size fractions at a distance y from the bed (grams per liter).
D = Geometric mean grain size of a given grain-size fraction (in feet).
d = Water depth (in feet).
I_1, I_2 = Definite integrals, functions of A and Z.
J_1, J_2 = Definite integrals, functions of A and Z.
N = The slope $\triangle V_y / \triangle (\log_{10} y)$ of the semilog plot of the velocity distribution.
p = Value of y at point of known concentration (in feet).
q_α = The water discharge per unit width in that part of the flow from the water surface

down to a distance α from the bed (in cubic feet per second per foot).

q_s = The suspended load for a certain grain size fraction per unit width of cross section (in pounds per second per foot).

$q_{s\alpha}$ = The measured load for a certain grain-size fraction per unit width of cross section in that part of the flow from the water surface down to a distance α from the bed (in pounds per second per foot).

q_{st} = The total suspended load for all sand grain-size fractions per unit width of cross section (in pounds per second per foot).

V = Average velocity in a vertical (in feet per second).

V_y = The velocity at a distance y from the bed (in feet per second).

$Y = y/d$ = a dimensionless distance from the bed.

y = Vertical distance from the bed (in feet).

y_1 = Value of y where the extrapolated velocity distribution gives $V_y = 0$ (in feet).

y_2 = Value of y where the extrapolated concentration distribution gives $C_{yt} = 480$ grams per liter (in feet).

Z = The slope $\triangle \log_{10} (C_y) / \triangle \log_{10} \frac{d-y}{y}$ of the logarithmic plot of the concentration distribution.

Semilogarithmic Velocity Distribution

When the velocity distribution in figure 2 is plotted on semilog paper, as in figure 3, the points fit a straight line reasonably well. The equation for this straight line is

$$V_y = N \log_{10} y + \text{const} \qquad (1)$$

Measurements in many rivers verify this semilogarithmic form of the vertical velocity distribution, and additional theoretical justification has been provided by Keulegan (*5*). With the velocity distributed as in equation 1, the *average* velocity in the vertical is given by

$$V = \frac{1}{d}\int_0^d V\,dy = \frac{N}{d}\int_0^d (\log_{10} y)\,dy + \int_0^d \frac{\text{Const}}{d}\,dy$$

$$V = N\,(\log_{10} d) - N/2.3 + \text{const} \qquad (2)$$

Combining equations 2 and 1,

$$V_y = N \log_{10} (y/d) + (V + N/2.3) \qquad (3)$$

Equation 3 describes any semilogarithmic velocity distribution. The parameters V and N can be graphically determined from a plot such as figure 3. N is the slope of the best-fitting straight line through the points. V can also be obtained from the line. To find the value of y where $V_y = V$, equate (2) and (3):

$$N \log_{10} y + \text{const} = N \log_{10} d - N/2.3 + \text{const}$$

$$\text{Log}_{10}\, d/y = 0.434$$

$$y/d = 0.368$$

Therefore, V is obtained from the line of best fit by reading V_y at a height 0.37 of the water depth from the bed ($y = 0.37d$).

The velocity distribution can be extrapolated below the lowest measured point by extending the straight line as in figure 3 or by using equation 3.

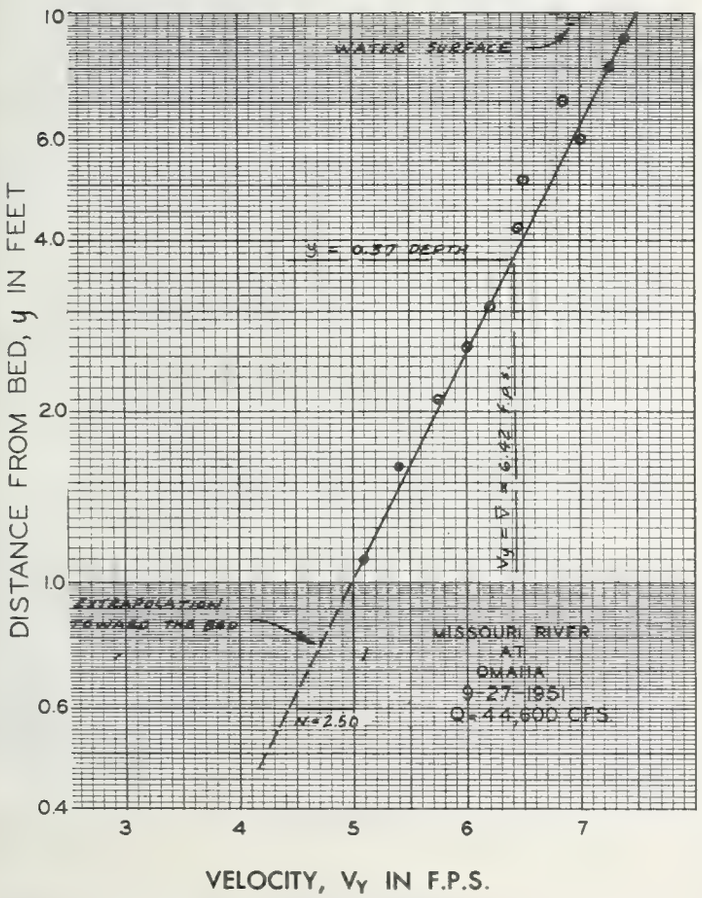


FIGURE 3.—Semilogarithmic plot of velocity distribution.

Concentration Distribution

We also find from field experience, with supporting evidence from the laboratory and some theoretical justification (*6*), that the vertical concentration distributions plotted on log paper in the form $\frac{d-y}{y}$ vs. C_y tends to be a straight line. This is the case in figure 4 which contains logarithmic plots for three of the grain-size fractions shown in figure 2. This form of the concentration distribution seems to hold not only for C_y, the concentration of individual grain-size fractions, but also for C_{yt}, the total concentration for all grain-size fractions in the sand range. The equation for these straight lines is in the form

$$C_y = C_p \left(\frac{p}{d-p}\right)^Z \left(\frac{d-y}{y}\right)^Z \qquad (4)$$

in which Z, C_p, and p are parameters obtained from the straight line that best fits the points plotted as in figure 4. Z is the slope of the line, and C_p is the concentration read from the line at a value of $\frac{d-p}{p}$ corresponding to a reference distance p from the bed.

The concentration distribution can be extra-

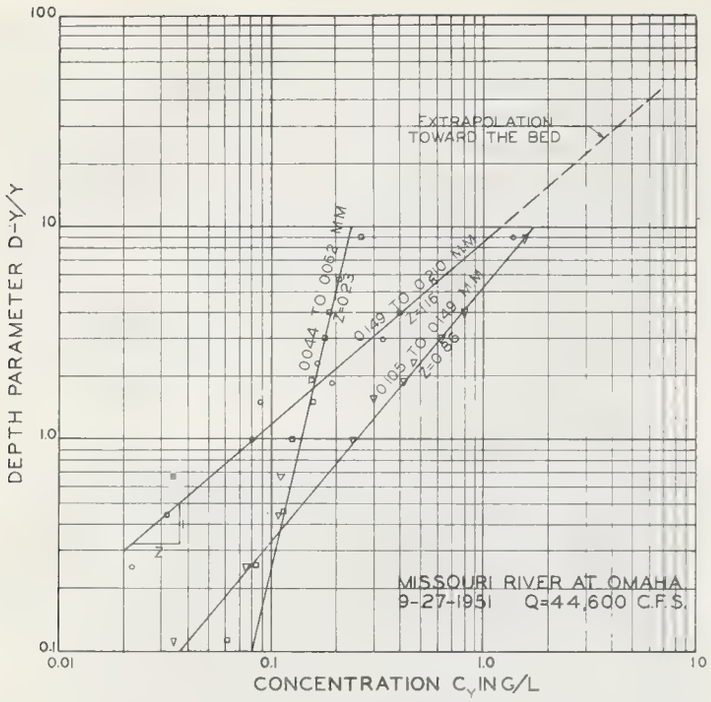


FIGURE 4.—Logarithmic plot of sand concentration distributions.

polated below the lowest measured point by extending the straight line as in figure 4 or by using equation 4.

Integrating to Get Suspended Load

The suspended load per foot of channel width for a given grain-size fraction is obtained by integrating the product of the velocity times the concentration over the depth in the vertical:

$$q_s=0.0624 \int_a^d V_y \cdot C_y \cdot d_y \qquad (5)$$

The choice of a, the lower limit of integration, will be explained later.

The above integration can be performed graphically, numerically, or analytically. In the graphical or numerical methods the velocity distribution and the concentration distributions for each grain-size fraction are plotted as in figures 3 and 4. The straight lines of best fit are then extrapolated below the measured points to the lower limit of integration, to $y=a$ for the velocity and to $\frac{d-y}{y}=\frac{d-a}{a}$ for the concentrations. By multiplying values of V_y and C_y at corresponding values of y, a third plot of V_yC_y vs. y is prepared. Finally the area under this curve between $y=a$ and $y=d$ is determined either planimetrically or numerically to obtain the unit suspended load as indicated in equation 5. It may be found that, due to the rapid changes in the values of V_yC_y near the bed, a logarithmic scale for y will be necessary. These methods are laborious, since separate integrations are required for each grain-size fraction for each sampled vertical in the river cross section. They are so time consuming, in fact, that they are impractical for routine use.

Einstein (*2*) has developed an analytical method of integration that reduces the time and labor required to the point where routine use is practical, especially when the method is programed for a digital computer. In preparation for this method, the vertical velocity distribution and the vertical concentration distributions for each grain-size fraction are plotted as in figures 3 and 4 and the straight lines of the best fit are drawn through the plotted points. The values of V, N, Z, p, and C_p are then read from these lines. These parameters are used to obtain q_s from a relationship that will now be developed. If we substitute equations 3 and 4 into equation 5,

$$q_s=0.0624 \int_a^d \left[N \log_{10} (y/d) + (V+N/2.3) \right] C_p \left(\frac{p}{d-p}\right)^Z \left(\frac{d-y}{y}\right)^Z d_y \qquad (6)$$

Let $Y=y/d$, then $dy=d\,dY$. At $y=d$, $Y=1$, and at $y=a$, $Y=\frac{a}{d}=A$.

$$q_s=0.0624\, N\, d\, C_p \left(\frac{p}{d-p}\right)^Z \left[\int_A^1 \left(\frac{1-Y}{Y}\right)^Z (\log_{10} Y)\, dY + (V/N+0.434) \int_A^1 \left(\frac{1-Y}{Y}\right)^Z dy \right] \qquad (7)$$

$$\text{Let } J_1=\int_A^1 \left(\frac{1-Y}{Y}\right)^Z dy$$

$$\text{and } J_2=\int_A^1 \left(\frac{1-Y}{Y}\right)^Z (\ln Y)\, dY$$

$$q_s=0.0624\, C_p\, N\, d \left(\frac{p}{d-p}\right)^Z \left[0.434\, J_2 + (V/N+0.434)\, J_1 \right] \qquad (8)$$

The integrals J_2 and J_1, plotted as functions of A and Z, are available in reference (*3*) for the ranges $0.00001 < A < 0.6$ and $0 < Z < 2.0$. For values of $2.0 < Z < 5.0$, we may use Einstein's integrals I_1 and I_2, plotted as functions of A and Z in reference (*2*). J_1 and J_2 are obtained from I_1 and I_2 are as follows:

$$J_1=4.63 \frac{(1-A)^Z}{A^{Z-1}} I_1 \qquad (9)$$

$$J_2=4.63 \frac{(1-A)^Z}{A^{Z-1}} I_1 \qquad (10)$$

If equations 9 and 10 are substituted in equation 8:

$$q_s = 4.63 \times 0.0624\, C_p\, Nd\, A\left[\frac{p}{A}\left(\frac{1-A}{d-p}\right)\right]^Z \left[0.434\, I_2 + (V/N + 0.434)\, I_1\right] \quad (8A)$$

Knowing the water depth, d, choosing a lower limit of integration, $A = a/d$, reading the parameters V, N, Z, p, and C_p from the measured velocity and concentration distributions, and obtaining values of J_1 and J_2 from references (*2*) or (*3*), we can compute the unit suspended load q_s from equation 8 for each grain-size fraction.

Defining Concentration Distributions With Less Samples

Figures 3 and 4 indicate that suspended samples and velocity measurements must be obtained at a number of points in the vertical in order to define adequately the parameters V, N, C_p, and Z. The velocities are relatively economical to obtain as they involve only fieldwork. The sediment samples, however, are relatively expensive, as they involve not only the fieldwork in obtaining them but also costly laboratory analysis that includes determination of the mechanical analysis of each sample as well as its concentration. In order to reduce the cost of processing the large volume of load measurements at the Missouri River stations, we sought a method of field sampling and office analysis that would minimize the number of suspended sediment samples required to define the concentration distributions.

In the method that was evolved we continued to measure enough point velocities to define V and N in a semilogarithmic plot such as figure 3. We obtained only two suspended samples, however:

(1) A point sample at a reference level p from the bed. From this sample, values of C_p are obtained for each grain-size fraction.

(2) A depth-integrated sample based on a traverse of the flow from the water surface down to a distance a from the bed. From this sample, values of C_a, the average concentration in the measured, depth-integrated part of the flow, are obtained for each grain-size fraction.

The actual field practice is to obtain two point samples and two depth-integrated samples and to combine each pair in order to obtain larger samples for laboratory analysis. The field measurements define values of d, V, N, p, C_p, a, and C.

The load q_{sa} for each grain-size fraction in the part of the flow that was depth-integrated is given by

$$q_{sa} = 0.0624\, C_a\, q_a \quad (13)$$

q_a is obtained from

$$q_a = \int_a^d V_y dy \quad (14)$$

Substitute the expression for V_y in equation 3 and integrate:

$$q = (d-a)\, V - N\, a \log_{10}\frac{a}{d} \quad (15)$$

q_{sa} can, therefore, be determined from equations 15 and 13, using known measured quantities.

Lower Limit of Integration

The question now arises how close to the bed to extend the velocity and concentration distributions in computing the suspended load. How do we choose a, the lower limit of integration? Inspection of equations 4 and 8 or 8A indicates that a must not equal zero or an infinite concentration at the bed, or infinite values of the integrals I_1, I_2, J_1, or J_2 would result. Three practical limitations on the choice of a come to mind:

(1) Extrapolation should be down to the upper edge of the layer near the bed where bedload supplants suspended load as the mode of transportation. For want of a better criterion, this is taken from Einstein (*2*) as $a = 2D$.

(2) The total concentration cannot be unrealistically high. Recent work of Einstein (*4*) and Bagnold (*1*) indicates that suspensions tend toward some limiting concentration, say 30 p.c.f. (480 g./l.).

(3) The velocity distribution should not be extrapolated below the value of y where $V = 0$, because equation 3 will yield negative velocities for smaller y values.

These limitations suggested the following procedure for choosing the lower limit of integration.

1. Compute the value of $2D$ for the particular grain-size fraction.

2. From a logarithmic plot of $\frac{d-y}{y}$ vs. C_{yt} for the total sand concentrations, obtain the values of Z, C_p, and p. Extrapolate this concentration distribution to find the distance from the bed, y_2, at which $C_{yt} = 480$ g./l. This may be done with a graphical extrapolation of the straight line of best fit or by using equation 4 to obtain

$$y_2 = \frac{d}{1+\left(\frac{480}{C_{pt}}\right)^{1/Z}\left(\frac{d-p}{p}\right)} \quad (11)$$

3. Extrapolate the semilogarithmic plot of the velocity distribution toward the bed to find the distance from the bed, y_1, at which $V_y = 0$. This may be done with a graphic extrapolation of the straight line of best fit or by using equa-

tion 3 to obtain

$$y_1 = \frac{0.37d}{10^{V/N}} \quad (12)$$

4. Choose $a = 2D$ or y_1 or y_2, whichever is the largest.

An expression for q_{sa}/C_p can be obtained from equations 8 or 8A:

$$q_{sa}/C_p = 0.0624\,N\,d\left(\frac{p}{d-p}\right)^z \left[0.434J_2 + (V/N + 0.434)\,J_1\right] \quad (16)$$

$$\text{or } q_{sa}/C_p = 4.63 \times 0.0624\,N\,dA\left[\frac{p}{A}\left(\frac{1-A}{d-p}\right)\right]^z \left[0.434I_2 + (V/N + 0.434)\,I_1\right] \quad (16A)$$

Knowing p and $A = a/d$ and assuming various values of Z, a relationship between Z and q_{sa}/C_p can be computed and plotted. The resulting graph can be entered with the values of q_{sa}/C_p determined from the field measurements, and the value of Z for each grain-size fraction can be read from the curve. When p is not equal to a, equation 16 or 16A may define two values of Z for a given value of q_{sa}/C_p. In this event Z values are read from the limb of the curve that gives ascending values of Z for q_{sa}/C_p values corresponding to ascending values of D.

After the Z values are determined by this method, the suspended load throughout the vertical is computed from equation 8 or 8A for each grain-size fraction.

Engineers with experience in measuring and computing suspended sand loads will recognize that this method, based on only two suspended load samples, cannot be as good as the more direct method of sampling a number of points to define the vertical concentration distribution. While theoretically, the results will be the same, practically, the precision is not comparable. Due to the variability of point concentrations, the more points that are sampled in a vertical in a short period of time, the better defined is the concentration distribution. On the other hand, regardless of the precision with which the distributions are defined in the measured part of the flow, we still face the uncertainty in extrapolating these distributions to the proximity of the bed. Since the estimated load must be indeterminate, regardless of the precision of the measurements, some economies in making these measurements would seem to be realistic. Although the above method of determining indirectly the Z values requires additional office analysis, the use of a digital computer in making the computations relieves this disadvantage.

This method gives a more reliable determination of suspended load than the modified Einstein method (*3*) developed by the U.S. Geological Survey, because it uses a direct measurement of the velocity distribution and uses enough suspended concentration data to define the vertical distributions without relying as heavily on theoretical relationships. The modified Einstein method does provide an estimate of the bedload as well as the suspended load. The method presented in this paper estimates the suspended loads only.

Examples

Example 1

The velocity and concentration distributions plotted in figure 1 were obtained at one vertical in the Missouri River at Omaha on 27 September 1951. From the plot of V_y vs. $\log_{10}y$ in figure 3, $N = 2.50$ and $V = 6.42$ f.p.s. For each grain-size fraction, $\log_{10}C_y$ was plotted against $\log_{10}\frac{d-y}{y}$ as in figure 4. The values of Z and C_p obtained from the best fitting straight lines are shown in table 1. The known concentration, C_p, was taken at mid-depth where $p = 5.0$ ft. and $\frac{d-p}{p} = 1$. The results of the suspended load computations for this vertical, with the use of equation 8A, are summarized in table 1. The ratios in the right-hand column show how much the suspended load exceeds the measured load for each grain-size interval and for the total sand. The values in the first line were based on a $\log_{10}C_{yt}$ vs. $\log_{10}\frac{d-y}{y}$ plot of the total sands. These are less than the values in the last line in table 1, obtained by adding the loads for the individual grain-size fractions.

Example 2

At station 3 + 65 on the Aksarben Bridge across the Missouri River at Omaha on 27 June 1957, the following data were obtained (table 2):

(1) A depth measurement of 16.0 feet.
(2) A vertical velocity distribution.
(3) A depth-integrated sample down to a point 0.5 foot from the bed.
(4) A point sample 8.0 feet from the bed.

A plot of V_y vs. $\log_{10}y$ gave $N = 3.00$ and $V = 5.77$ f.p.s. Laboratory analyses of the two sediment samples yielded the values of C_a and C_p shown in table 3. A Z vs. q_{sa}/C_p relation was computed from equation 16A. The results are summarized in table 2 and plotted in figure 5.

The determination of the Z values in the sampled vertical is shown in table 3. From the measured values of C_a and C_p, values of q_{sa}/C_p are computed for each grain-size interval. The corresponding Z values are then read from figure 5.

TABLE 1.—*Suspended sand load computation from point samples at station 3+65 of Missouri River at Omaha, 27 September 1951*

$N=2.50$; $V=6.42$ f.p.s.; $d=10.0$ ft.; $p=5.0$ ft.; $\alpha=1.0$ ft.;

$$y_1=\frac{0.37\times10.0}{10^{6.42/2.50}}=0.010 \text{ ft.}; \quad y_2=\frac{10.0}{1+\left(\frac{480}{0.82}\right)^{1/0.72}}\frac{10.0-5.0}{5.0}=0.00143 \text{ ft.}$$

Size interval	Mean size D	Z	C_p	Computed suspended load							Computed measured load				Ratio
				$2D$	Lower limit α	$A=\frac{\alpha}{d}$	I_1	I_2	q_s	$A=\frac{\alpha}{d}$	I_1	I_2	$q_s\alpha$	$\frac{q_s}{q_s\alpha}$
Mm.	*Mm.*		*G./l.*	*Ft.*				(−)	*Lb./sec./ft.*			(−)	*Lb./sec./ft.*	
Total sand	0.185	0.72	0.820	0.00121	y_1	0.0010	3.616	10.29	5.46	0.10	0.4878	0.6165	3.448	1.59
0.044–0.062	.053	.23	.139	.00035	y_1	.0010	47.72	68.02	.559	.10	1.1444	1.0603	.495	1.13
0.062–0.074	.068	.55	.058	.00045	y_1	.0010	7.980	18.09	.301	.10	.6325	.7309	.0222	1.36
0.074–0.105	.099	.58	.270	.00065	y_1	.0010	6.880	16.26	1.456	.10	.6027	.7084	1.047	1.39
0.105–0.149	.127	.86	.240	.00083	y_1	.0010	2.065	6.935	2.099	.10	.4031	.5421	1.117	1.88
0.149–0.210	.179	1.16	.082	.00117	y_1	.0010	0.812	3.579	1.579	.10	.2837	.4295	.505	3.12
0.210–0.297	.254	1.39	.0031	.00167	y_1	.0010	0.488	2.460	.132	.10	.2263	.3602	.025	5.32
Total or average									6.126				3.412	1.80

TABLE 2.—*$q_{s\alpha}C_p$ computed for various values of Z at station 3+65 of Missouri River at Omaha, 27 June 1957*

$N=3.00$; $V=5.77$ f.p.s.; $d=16.0$ ft.; $p=8.0$ ft.; $\alpha=0.5$ ft.

Assumed Z	$A=\alpha/d$	I_1	I_2 (−)	$q_s\alpha/C_p$
0	0.03125	6.7081	5.9593	5.73
0.375	.03125	2.0820	2.7992	5.76
0.575	.03125	1.2710	2.0166	6.59
0.790	.03125	.80812	1.4852	8.25
1.10	.03125	.45209	.9318	12.90
1.26	.03125	.38223	.8771	17.1

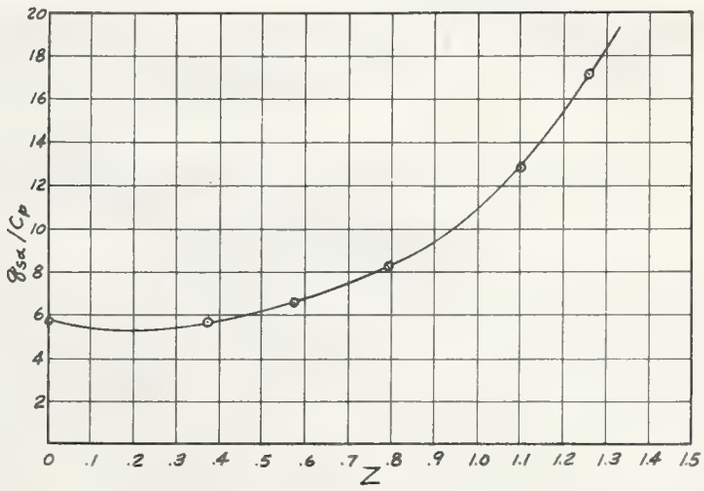


FIGURE 5. Z vs. $q_s\alpha/C_p$ plot for example 2.

With the use of the Z values determined in table 3, the load for each grain-size interval can be computed as in example 1. It can be seen from table 3 that considerable judgment must be applied in selecting the Z values and that the method will fail if measured concentrations are too low for adequate computation of the needed parameters.

TABLE 3.—*Determining Z from measured q_s/C_p values at station 3+65 of Missouri River at Omaha, 27 June 1957*

$N=3.00$; $V=5.77$ f.p.s. $d=16.0$ ft.; $\alpha=0.5$ ft.; $q\alpha=(16.0-0.5)$

$5.77-3.00\times0.5\log_{10}$; $0.5/16.0=91.6=\frac{\text{c.f.s.}}{\text{ft.}}$

Size interval	Mean size D	$C\alpha$	C_p	q_s	$\frac{q_s\alpha}{C_p}$	Z (from fig. 5)
Mm.	*Mm.*	*G./l.*	*G./l.*	*Lb./sec./ft*		
0.044–0.053	0.049	0.115	0.098	0.657	6.70	[1]0
0.053–0.074	.063	.142	.132	.811	6.15	0.50
0.074–0.105	.099	.154	.055	.881	16.0	1.22
0.105–0.149	.127	.0426	.0422	.243	5.75	([2])
0.149–0.210	.179	.0033	.0063	.0189	3.00	([2])
Total Sand		.458	.365	2.62	7.19	.66

[1] Point falls to left of $Z=0$ on left limb of curve. Z taken as zero as would be expected of such a small-grain size in this high a velocity.

[2] Measured concentration too low to provide a good estimate.

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DESIGN PRACTICE FOR LEVEE REVETMENT ON WEST COAST INTERMITTENT STREAMS

[Paper No. 57]

By A. P. GILDEA, *chief, Hydraulics Section, U.S. Army Engineer District, Los Angeles*

Synopsis

One of the early attempts to evaluate the effectiveness of riprap revetment to protect the banks of intermittent streams flowing at velocities approaching the critical value was made by the U.S. Army Engineer District at Los Angeles after a major flood occurred in the area in March 1938. Unfortunately, record surveys of conditions before the flood were not sufficient to compute accurately the maximum scour during the flood. Because a substantial amount of the revetted levees and banks were destroyed by the flood, drastic design modifications were made for rehabilitation after the flood.

Since 1950 there has been a progressive development of design criteria for revetment to resist the erosive forces of velocities in the range between 10 and 18 feet per second. In 1952, the Office, Chief of Engineers, issued Civil Works Engineer Bulletin 52-15 in order to standardize levee slope protection works. In recent years the Los Angeles District of the Army Corps of Engineers has built levees on three California streams that have only intermittent discharge. The design of revetment is different for each. These designs are delineated so that current practice may be appraised. To protect from underscour, the toe of the revetment should be carried to a point slightly below maximum depth of anticipated scour by the design flood. This point may be 6 to 12 feet below the thalweg of the stream.

Introduction

During the mid-1930's emphasis was placed on public works in order to bring employment to depressed areas. The big problem, however, was that the entire United States was a depressed area. Thus, flood control practices in the form of local protection projects were established by the Congress. On the West Coast one of the first such projects authorized was that for the Los Angeles River, which flows through the metropolitan area of the city of Los Angeles. As part of the project work, the banks of the Los Angeles River and its tributaries were revetted with dumped and hand-placed stone obtained from a local quarry. These were the days when the term "one man stone" had meaning.

After completion of many miles of paving of the Los Angeles River, a major flood occurred in the area in March 1938. Much of the channel revetment resisted the flood flows, but unfortunately levees and banks were destroyed in many places. The revetment destroyed by the flood appeared to have failed as a result of underscour of the levee toe. A total of 15 miles of hand-laid revetment stone had been placed on a side slope of 1 on 3 and controlled by two specifications:

(1) Type A for concave side of bends with curvature having radii of 3,500 feet or less. Stone consisted of 12- to 16-inch pieces placed on a 6-inch blanket of spoils. Thickness of stone layer was generally 12 inches, except at streambed, where a heavier section was made up of 16- to 24-inch quarry stone extended to a depth below grade.

(2) Type B for straight levees, for the convex side of all bends and for the concave side of bends with a curve radius greater than 3,500 feet (fig. 1).

Types of bank protection used by local agencies in the area consisted of wire and rock mattress, gunite slope paving, asphalt slope paving, and loose stone paving. A total of 23 miles of channel revetment consisted of heavy stone dumped in place on a slope of about 1 on 2 (fig. 2). About 30 miles of channel had banks protected by pipe and wire fence as shown in figures 3 and 4.

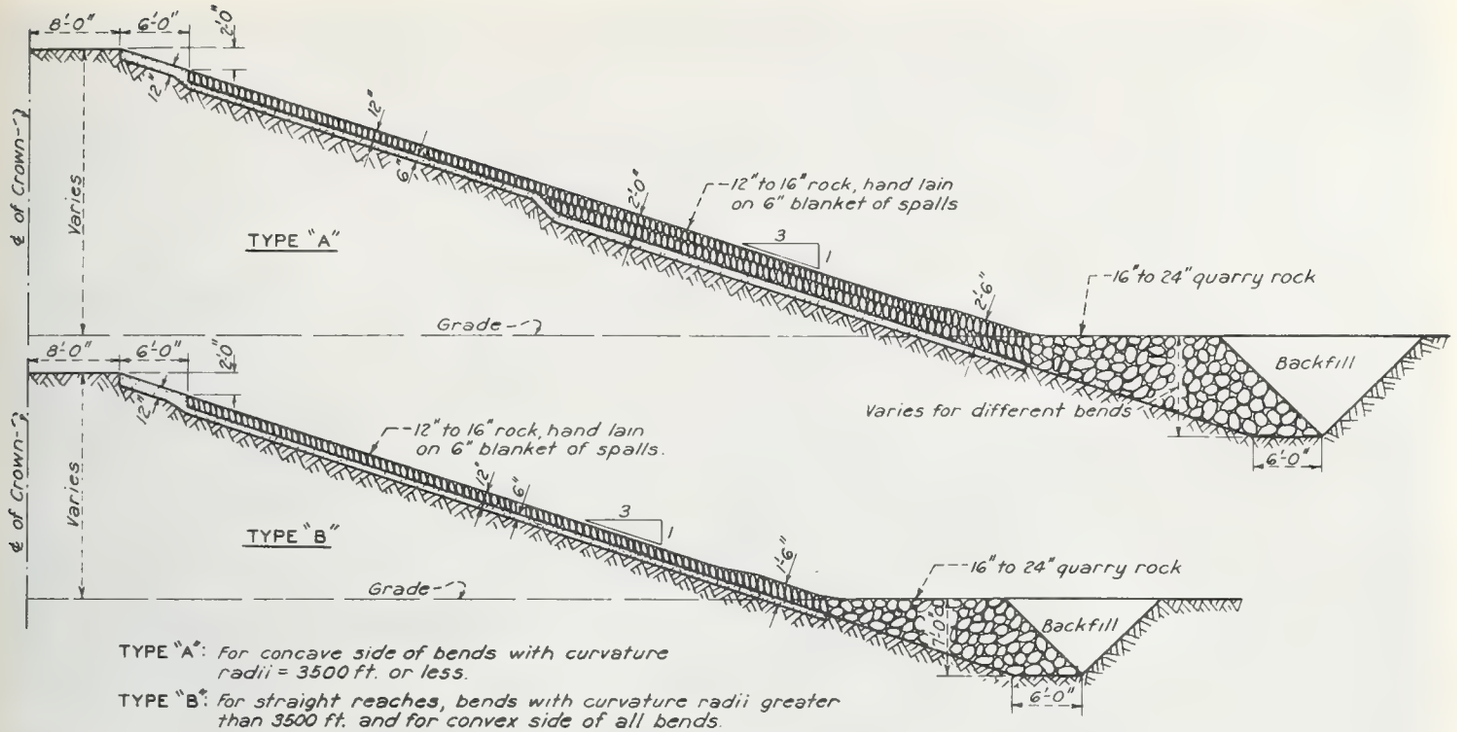


FIGURE 1. — Rock paving bank protection; typical section.

Damage Resulting from 1938 Flood

The upper reach of Los Angeles River northwest of the city was not completed as a whole at the time of flood. The channel in the vicinity of Griffith Park was designed to carry a discharge of 40,000 c.f.s. on a bottom slope varying between 0.0037 and 0.0040. An estimated peak discharge of 60,000 c.f.s. occurred in this reach on March 2, 1938, with velocities in excess of 20 feet per second. The failures on the upper Los Angeles River may be attributed directly to poor channel alinement, disturbances created by side inflows, and ponding of local drainage behind levees. Exploration by test pits and probing at the sites of some failures disclosed that 18 inches of the top layer of toe rock had been removed, but that otherwise the deeper rock had not been disturbed. There were some indications that failure may have commenced immediately above the toe rock. Other failures, especially on curves, seem to have commenced by underscour of the levee toe, the stone at this point having been moved out, and water entered the fine sand of levee fill, undercutting the upper

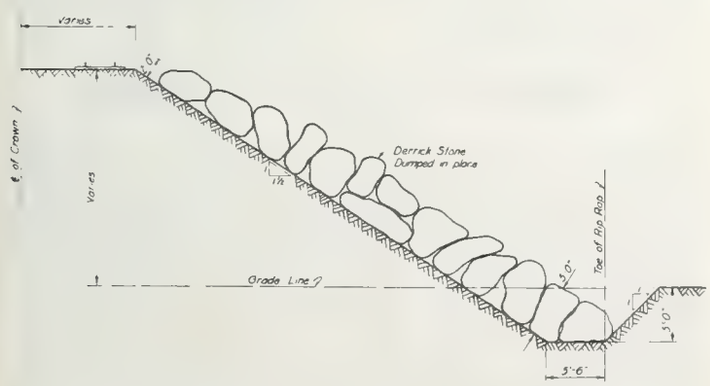


FIGURE 2. — Riprap bank protection; typical section.

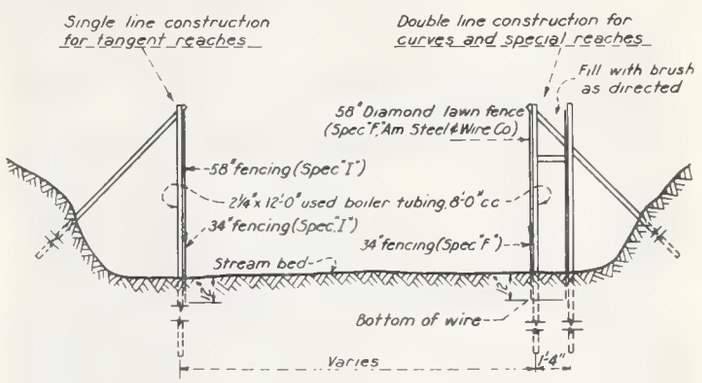

Single line construction for tangent reaches
Double line construction for curves and special reaches
Fill with brush as directed
58" Diamond lawn fence (Spec "F" Am Steel & Wire Co)
58" fencing (Spec "I")
2½" x 12'-0" used boiler tubing 8'-0" cc
34" fencing (Spec "I")
34" fencing (Spec "F")
Stream bed
Bottom of wire
Varies
1'-4"

FIGURE 3. — Pipe-and-wire bank protection; typical section.

section. Failure progressed downstream until a break of sufficient capacity to bypass the levee was made, or until the current was deflected away from the levee. The velocity of flow in this reach of the river probably equaled critical velocity with attendant undulations until the channel banks were breached. Figure 5 shows an aerial view of damaged levees as a result of

FIGURE 4. — Typical pipe-and-wire fence used for bank protection. Note how channel bottom is eroding and undercutting fence along left bank.

sinuous channel alinement. Figure 6 shows two photographs; one indicating destruction of one bank of the river as a result of concentrated side inflow from the opposite bank; the other showing failure along the outside of a bend. Figure 7 shows photographs of destruction of a railroad bridge as a result of one pier foundation being undercut by scour.

The downstream reach of the channel southeast of the city of Los Angeles carried a peak discharge estimated at 63,000 c.f.s., with the maximum velocity of approximately 17 feet per second. The slope of the bottom was 0.0025. Although the flood was of sufficient magnitude to destroy partially or totally seven railroad or highway bridges and to do considerable damage to the bank in several locations, no appreciable damage occurred to the 2.4 miles of straight rock-paved channel, again indicating that channel alinement was a major factor in the performance of the revetment. Also the velocity of flow was definitely less than critical velocity. Photographs of riverbank dumped stone revetment show failure probably resulting from underscour along the levee toe (fig. 8). Figure 9 shows a straight reach where bridges were destroyed but the bank revetment remained intact throughout the flood.

The pipe and wire fence type of protection used extensively along Rio Hondo and San Gabriel River played an important part in preventing bank erosion and confining the flood within channels in straight or nearly straight reaches. Large sections failed, however, on the outside of bends or where some obstruction deflected the main current against the fence. The weakness of this form of protection was the inability to deflect cross flows sufficiently to prevent bank erosion back of the fence and the ease with which pipe posts were broken off or pulled out when encountered by the full force of the current. Figure 10 shows photographs of bank erosion and destruction of pipe and wire fence type of protection.

Transportation and Deposition of Sediment

After the March 1938 flood, approximately 36 miles of the Los Angeles River from a point in the San Fernando Valley to the Pacific Ocean were examined in an effort to determine the magnitude of scour. However, between the date of the previous survey and the date of the flood, considerable construction work had been carried out in several reaches. For the most part this consisted of removal of material from the

FIGURE 5. — Upstream view of damage to Los Angeles River channel in 1938 flood: Maximum discharge 60,000 c.f.s.; maximum velocity about 20 f.p.s.

riverbed for levee construction. This work undoubtedly had some effect on the sediment movement, but its importance cannot be determined. The change in volume in several typical reaches not affected by construction was computed. That is, computations of "scour" or "deposit" were attempted. In many cases scour of both banks and bottom occurred. In other cases where the entire levee was eroded away, the scour in the riverbed could only be estimated.

It is recognized that the above measurements indicate only the net change in section due to the storm. It is probable that many reaches of channel were scoured to a considerable depth on the rising stage and refilled to practically the original elevation on the falling stage. In such cases measurements after the flood would erroneously indicate little or no movement of sediment during the flood. To obtain an indication of the maximum scour, probings to a solid bottom were made in several locations. On the Los Angeles River immediately upstream from Los Feliz bridge, 14 cross sections were surveyed and probed. Thus the quantity of deposit after the maximum scour could be estimated. No satisfactory cross section studies before the storm were available. Therefore, the maximum scour and net change of section could not be determined. As for the ground surface formation in the streambed and banks, it would be classed as recent alluvium consisting chiefly of silt, sand, and gravel. Examination of the plot of centerline profiles measured before and after the storm leads to the conclusion that the average scour was about 6 feet (fig. 11). Width of channel at this point was 190 feet. The flood discharge at this location was estimated at 60,000 c.f.s., with mean velocity of about 13 feet per second.

Near the mouth of the Los Angeles River probings were made at two cross sections 200 feet apart. The probings and the cross sections

FIGURE 6.—Upstream views of Los Angeles River channel after 1938 flood: *A*, Right bank destroyed downstream from Verdugo Wash confluence; *B*, left bank damaged along outside of bend.

indicate a scour of 15,400 cubic yards per station before the peak scour, and a deposit of 13,530 cubic yards between the time of the peak scour and 14 March, about 12 days later. An additional deposit of 1,270 cubic yards per station occurred in the subsequent month. The width of channel at this point is 540 feet, and the peak discharge was about 75,000 c.f.s. with a mean velocity of about 12 feet per second. One final effort was made to obtain a reasonable estimate of total volume of sediment transported from the Los Angeles River by means of a hydrographic survey of the Pacific Ocean at the mouth of the river. From a study of aerial photographs and soundings, it was estimated that 6,600,000 cubic yards of sediment was deposited by the flood.

Design Modifications Resulting From 1938 Flood

The flood of March 1938 demonstrated that the type of rock toe constructed along the river prior to the flood was vulnerable to high velocity currents. That is, the revetment was weak at the junction of the slope and the invert grade and the size of the rock in the toe protection section was too small. Therefore, a complete new design was developed for rehabilitation of the river after the flood. All remaining undamaged stone revetment was grouted, and where levee sections had to be rebuilt the slope paving consisted of a 12-inch thick concrete slab.

Levee Toe Protection

To protect the levee slope paving from being undercut by the high velocity current, a heavy toe section made up of derrick stone varying in size from 1 to 3 tons was developed. The depth of the toe protection extended 7 feet below

FIGURE 7. — Railroad bridge crossing Los Angeles River destroyed during 1938 flood as result of underscour of pier: *A*, High discharge during flood; *B*, after flood receded.

FIGURE 8. — Downstream views of Los Angeles River showing bank failure after dump stone was undercut and washed away.

streambed, and the voids between the stone were filled with concrete. Along the outside of sharp curves, the toe was extended 9 feet below streambed and 20-foot long steel sheet piling was driven under the toe for added protection. Details of this design are shown in figure 12. Placement of the heavy stone along the toe of levee is shown in figure 13. When the voids in the toe stone were filled with concrete, the entire section was made massive but also made rigid. The 9-foot depth of toe would allow 9 feet of streambed degradation, but if the degradation exceeded 9 feet, the heavy toe would be undermined and probably would fall into the scour hole. Also filling the voids with concrete produced a relatively smooth and rigid boundary that in turn induced scour along the levee toe. Figure 13 shows photographs of the toe stone being placed along the banks of the Los Angeles River channel. The photographs also show the character of the banks and bottom material, which was made up of fine sand and silt.

Use of Stabilizers

In the Los Angeles River, the use of transverse sills, called stabilizers, were placed at intervals of about 3,000 feet to act as controls for preventing progressive streambed degradation. During the period 1939–44, flood discharges in the river were moderate, the maximum discharge being about one-third of the design value. A survey of streambed conditions in 1944 showed that local scour at the stabilizers had progressed to the point where extraordinary maintenance work was required. To resist further degradation, heavy stone had to be placed downstream from the stabilizers and along the exposed toe of the levees for a considerable distance downstream. Figure 14 illustrates the eroded condition of the streambed in the vicinity of a typical stabilizer. Note also the exposed toe of levee for a considerable distance downstream. The smooth surface produced by the grouted stone induces scour along the rigid boundary. The experience with stabilizers on Los Angeles River indicated that the introduc-

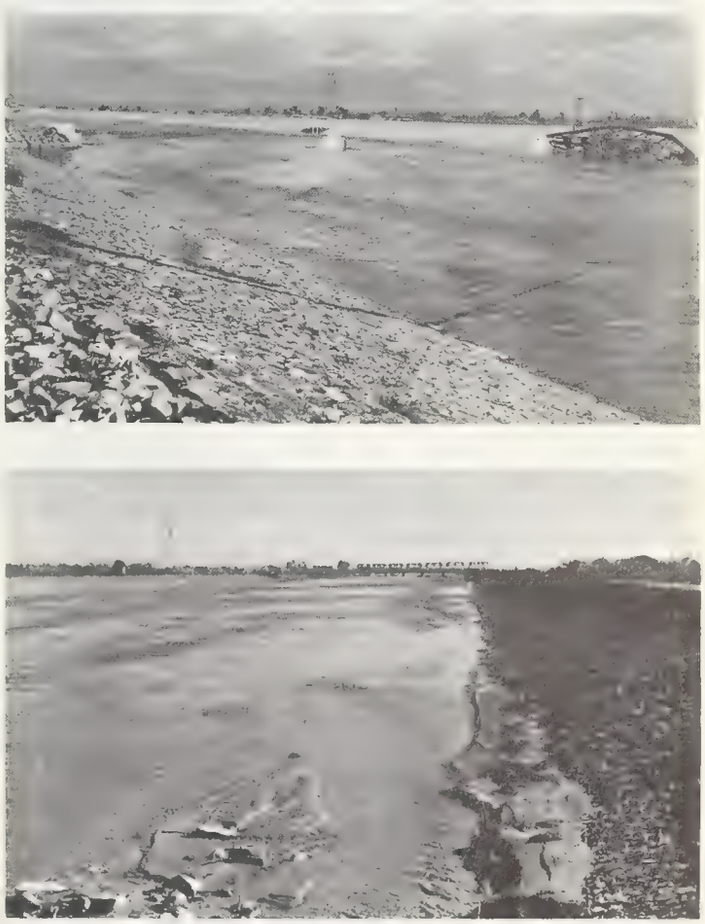


FIGURE 9. — Straight reach of lower Los Angeles River where handplaced stone withstood the flood. Note bridge truss destroyed in upper photograph.

FIGURE 10. — Views showing inadequacy of pipe-and-wire fence to protect against bank erosion.

tion of a rigid boundary in the form of a transverse sill produced an aggravated local scour condition that might produce levee failure more rapidly in any one flood than would general streambed degradation.

Use of Grouted Cobblestone

On streams such as the upper reaches of San Gabriel River near the mountains and the Ventura River in which the streambed material is made up of a large percentage of cobblestones, advantage was taken of this material and a levee-revetment design based on the use of cobblestones was developed. This design specified a layer of dumped stone 18 inches thick at the top of the levee and 24 inches thick at the toe, and the voids filled with sand-cement grout. The toe of the stone layer was placed at a depth 8 feet below the thalweg of the stream. The excavated toe trench was backfilled with random streambed material. Cost of this type of levee was less than importing quarry stone large enough to resist the velocity that would be 16 to 18 feet per second. Figure 15 shows the design details for this type of revetment. Figure 16 shows characteristics of streambed material and dumped stone before and after placement of grout. Only moderate floods have occurred since completion of construction of these two levees; therefore, no experience is available to demonstrate the effectiveness of this type of construction.

Field Research to Support Design Assumptions

West Coast districts of the Corps of Engineers have made studies to determine adequacy of riprap revetment for protection of levee banks against erosion. The usual practice was to use large particles with corresponding large voids. For example, in the Portland Distric field observations in 1949 showed that an 18-inch layer of dumped stone consisting of individual pieces weighing from 50 to 300 pounds is generally adequate for streams having maximum observed bank velocities of approximately 10 feet per second, provided the toe section is properly designed to prevent undercutting. The general practice in this regard was to carry the bank revetment to a depth 5 feet below the thalweg of the stream.

Civil Works Investigations

In 1951 the Portland district conducted high velocity revetment tests in the outlet channel immediately downstream from Dorena Dam in the Willamette Valley. Four sizes of rock revetment were tested against velocities ranging from 5 to 23 feet per second. The test sections had the advantage of being conducted under prototype conditions, with flows in the test channel controlled by regulating the outlet gates of Dorena Dam. The bottom of the 200-foot long test channel was grouted to prevent undercutting at the toes of the revetment slopes. The bank for the test sections had a slope of 1 on 2. During the initial stages of the tests the downstream end of the test channel eroded severely and a grouted rock rib was constructed at that point. The results of the tests in general showed that rocks ranging from 3 to 130 pounds in size, with an average weight of 20 pounds, resisted attack velocities adjacent to the bank from 6 to 11.5 feet per second. Rock ranging in size from 3 to 200 pounds, with an average size of 40 pounds, resisted attack velocities of 5.5 to 9 feet per second and probably would have resisted considerably higher velocities. Rock ranging in size from 7 to 700 pounds, with an average size of 80 pounds, failed with an average attack velocity of 13.3 feet per second. Also, rock in the range from 8 to 700 pounds with an average

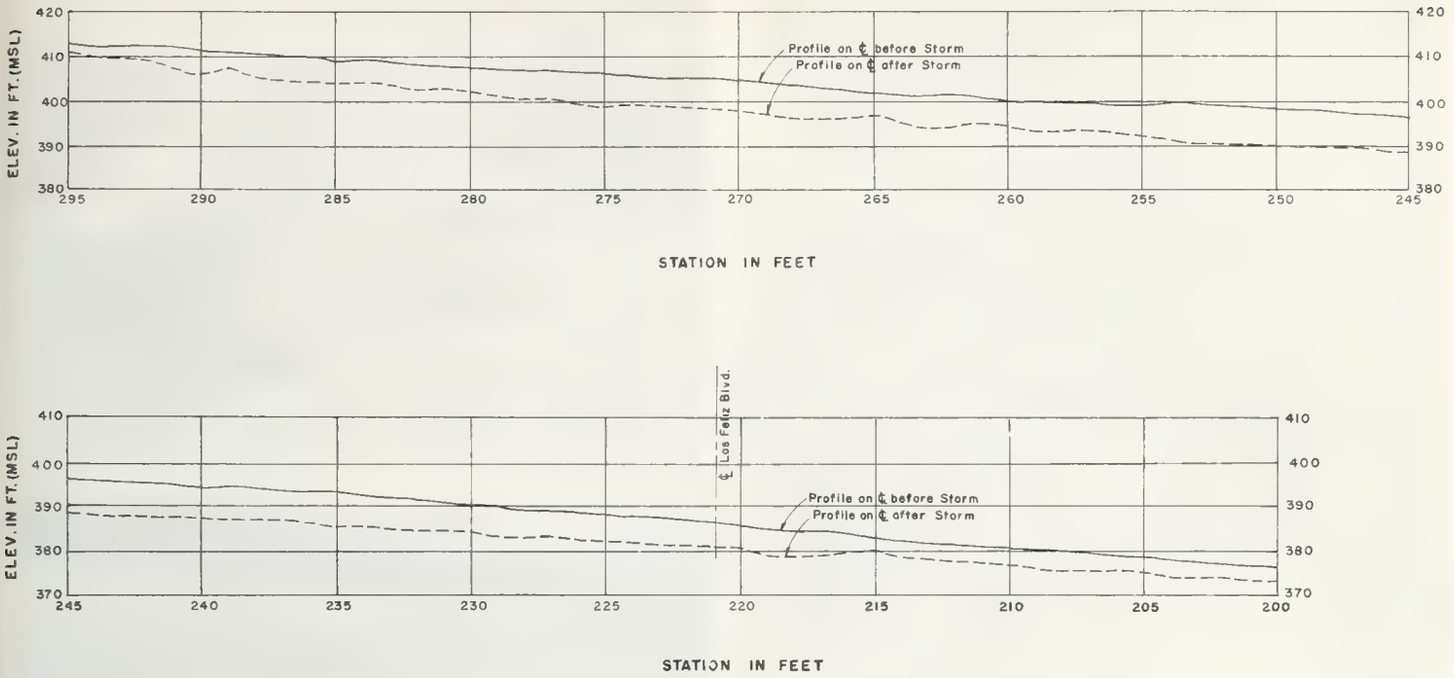


FIGURE 11. — Los Angeles River streambed profile before and after flood of March 1938.

size of approximately 100 pounds failed with an average attack velocity of 16.5 feet per second. The thickness of the slope protection tested ranged from 12 to 24 inches.

Between 1949 and 1951 the Seattle District conducted a civil works investigation to study the minimum riprap requirements for protection against high velocity flow. The investigation involved construction of test sections with various sizes of riprap along the Green River,

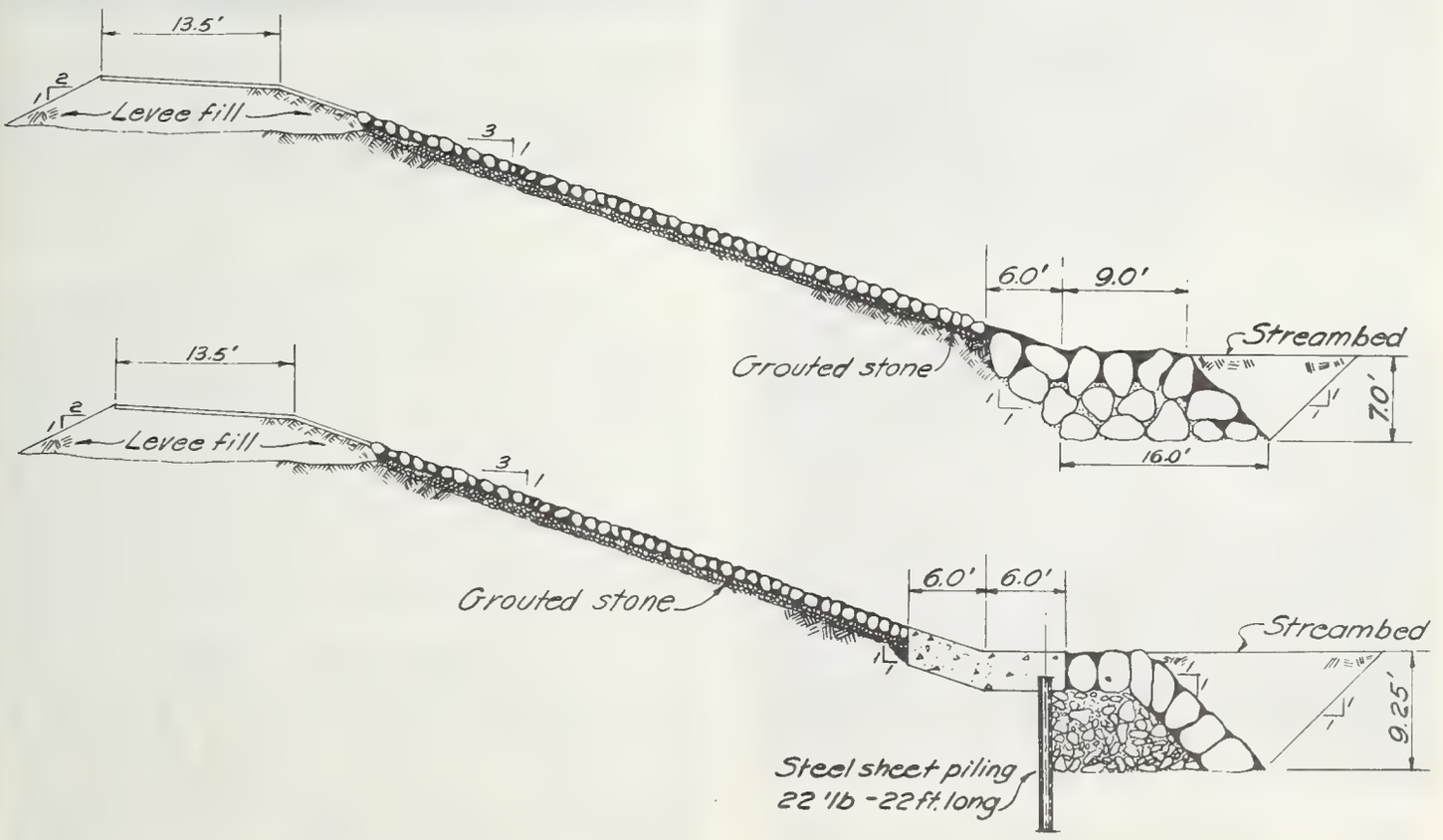


FIGURE 12. — Los Angeles River typical levee revetment; toe design used after 1938 flood.

which is a high velocity stream. A total of 6 rock sections were constructed with average rock size ranging from 20 to 400 pounds. The thickness of the revetment on the various sections ranged from 12 to 24 inches and the various sections were constructed on 1-on-2 slopes. In addition, one section was constructed with cobblestone ranging in size from 4 to 12 inches and the thickness of the revetment ranging from 16 to 19 inches. During a high water period in February 1951, observations and channel and bank velocity measurements were made at the test sections. The results of the rock revetment tests indicate that a 12-inch blanket of riprap in which 50 percent of the rock weighs between 20 and 90 pounds will provide protection against velocities of up to 10 feet per second and that an 18-inch thick riprap blanket consisting of rock averaging 100 pounds or over is necessary to provide protection against current velocities of up to 12 feet per second.

FIGURE 14. — Eroded streambed in vicinity of stabilizer: *A*, Scour at stabilizer for skew railroad bridge is about 6 feet and diminishes to 3 feet in a downstream distance of 80 feet; *B*, along right levee toe, scour is maximum of 5 feet and diminishes to 3 feet in a downstream distance of 500 feet.

FIGURE 13. — Heavy toe stone being placed by derrick along banks of Los Angeles River channel.

Engineer Bulletin 52-15

As a result of these civil works investigation projects involving an appraisal of existing slope protection on river banks and levees and the special test sections, the Office, Chief of Engineers issued Civil Works Engineer Bulletin 52–15, entitled "Slope Protection," dated 2 June 1952. This bulletin gives riprap thickness, rock size, and approximate grading for various ranges of channel velocities. In the bulletin the required thickness and size of rock are based on the velocity within 10 feet of the bank. The bulletin provides that where hydraulic dredge tailings are readily available, a 12-inch layer of cobbles with a maximum size of 10 to 12 inches is satisfactory for bank velocities up to 8 feet per second. The bulletin also provides that where channels are formed by banks or levees of fine sands or silts that are readily erodible, protection by a layer of gravel or crushed stone

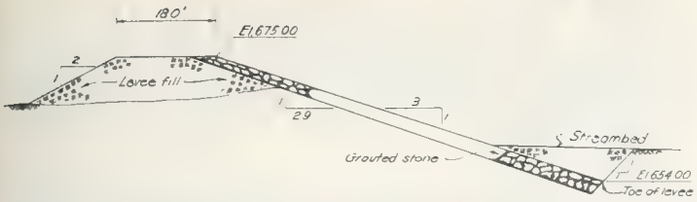


FIGURE 15.—Ventura River typical levee revetment of grouted cobblestone.

8 to 12 inches in thickness will be required for velocities between 3 and 6 feet per second. The bulletin further states that in most cases, riprap should be terminated in a rock toe at the level of the streambed to prevent undermining the channel protection. The toe should have a minimum base width of 6 feet and a minimum thickness of 3 feet with 1-on-1½ side slopes. Where the channel is composed of sand or silt, it is considered better practice to carry the riprap down below the channel level to a minimum depth of 5 feet vertically and omit the rock toe. On large rivers, or tidal estuaries, having a considerable depth of flow at low water stages, the riprap need be carried down only 5 feet vertically below mean low water. In such cases a rock toe is unnecessary. The recommended stone gradation and thickness is given in table 1:

TABLE 1.—*Recommended sizes of stone riprap for indicated velocity*

Current velocity (ft./sec.)	Riprap thickness	Maximum size	Riprap grading		Filter thickness
			Average size	20 percent size	
	Inches	*Pounds*	*Pounds*	*Pounds*	*Inches*
6 to 12.	12	150	30 to 50..	20	6
12 to 15.	15	250	50 to 80..	20	6
15 to 18.	18	400	80 to 150.	20	6

Many offices of the Corps of Engineers have had difficulty in using the criteria in Civil Works Engineer Bulletin 52–15 because the specified velocity 10 feet from the bank during design flood stages is very difficult to measure in the field and cannot be reliably computed. However, the criteria given in the bulletin are considered as a guide rather than as hard and fast specifications. Deviation from Civil Works Engineer Bulletin 52–15 appears desirable, particularly when the levee follows sinuous alinement.

Recent Design Practice

The various field observations and research investigations appear to have established adequate design criteria for size of revetment stone and thickness of layer to withstand various velocities up to a maximum of about 15 feet per second. However, to date little knowledge has been gained regarding the depth to which the levee revetment must be carried to be safe against scour. Unquestionably, the maximum scour depends upon a number of variables, such as the fineness of the streambed material, alinement of the boundary of the stream, gradient and velocity of stream, and source of supply of sediment.

In recent years the Los Angeles District of the Corps of Engineers has constructed levees of three West Coast streams; namely, Santa Ana River in Riverside County, Santa Clara River in Ventura County, and Santa Maria River in Santa Barbara County, Calif. On each of these rivers, the most controversial feature of the design was the required depth of the levee toe.

On the Santa Ana River in the vicinity of the city of Riverside, levees were designed and constructed to protect the local area from flood flows having near critical velocities of about 18 feet per second. The design discharge and depth of flow would be 195,000 c.f.s. and 15 feet, respectively. Figure 17 shows an aerial view of partial improvement in which a single levee has

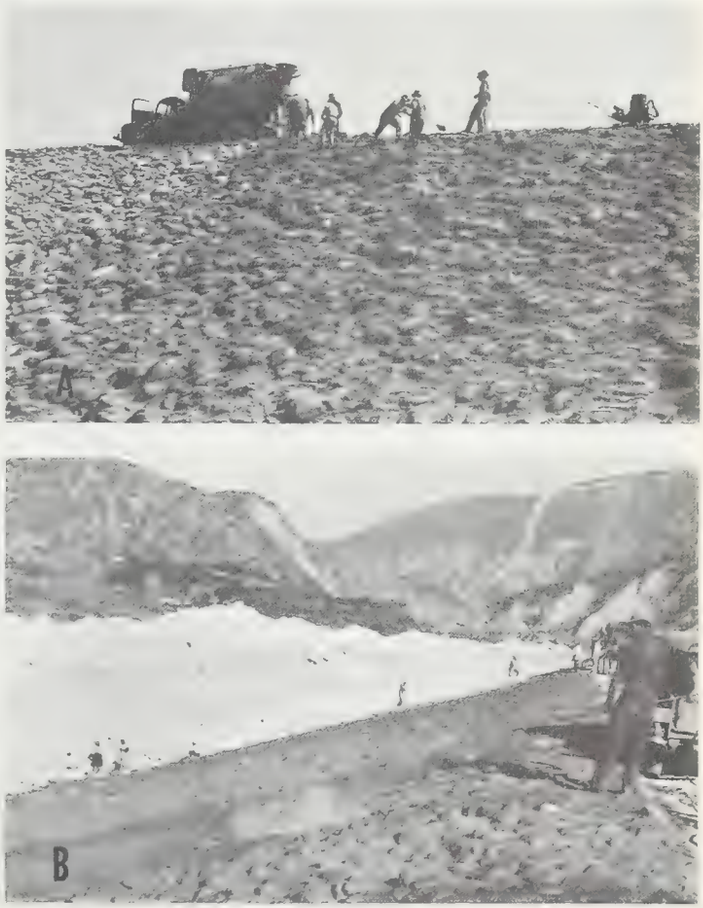


FIGURE 16.— Cobblestones obtained from streambed are graded and placed on levee, then grouted: *A*, Ventura River levee; *B*, San Gabriel River levee. Note character of streambed material in toe trench excavation.

been completed upstream from Crestmore Road. Double levees were subsequently built downstream from Crestmore Road. The details of the levée design are shown on figure 18. Maximum size stone used on the levee was 400 pounds with 50 to 70 percent being 150 pounds or less. The maximum toe stone size was 2,000 pounds and 60 percent ranged between 400 and 1,000 pounds. The toe rock was placed 11.5 feet below streambed because of the anticipated

FIGURE 17. — Upstream view of Santa Ana River in Riverside County, Calif., showing partial completion of the project. Double levees were subsequently built downstream from Crestmore Road to confine a discharge of 195,000 c.f.s.

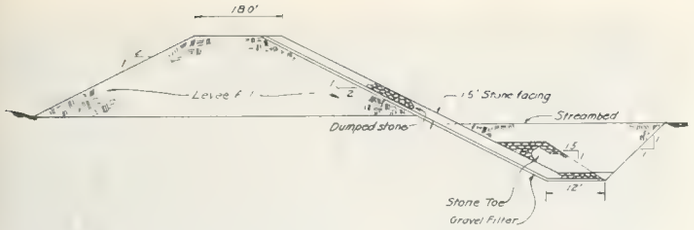


FIGURE 18.—Santa Ana River typical levee section.

scour during the design flood. Streambed material consisted of medium and fine sand and silt. The slope of the bottom of the stream was 0.0042.

On the Santa Clara River a single levee was designed and constructed to protect the coastal plain, in the vicinity of Oxnard, from floodflows having subcritical velocities of about 14 feet per

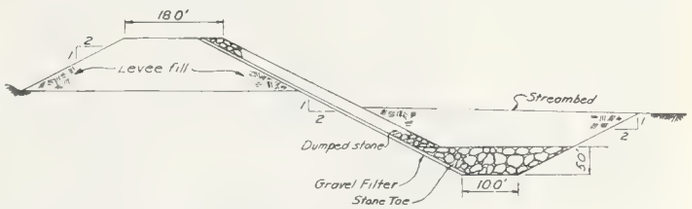


FIGURE 19.— Santa Clara River typical levee section.

FIGURE 20.— Upstream view of Santa Clara River in Ventura County, Calif., showing a single levee designed to confine a discharge of 225,000 c.f.s.

second. The river was confined on the right side by a high natural bank. For the design flood of 225,000 c.f.s., the depth of flow would be about 15 feet. The stream gradient was 0.0030. Sand and gravel mining was being carried on in the streambed by private land owners, and it appeared that the open pits remaining would mitigate against the possibility of maintaining current directions parallel to the levee. Thus, heavy stone at the toe of the levee appeared necessary to withstand impinging attacks by the current. Maximum size stone used on the levee was 400 pounds and 50 to 70 percent were 150 pounds or less. Maximum toe stone size was 1,000 pounds and the median size was about 400 pounds. The toe stone was placed 8 feet below the thalweg of the stream. The design details of a typical levee section are shown in figure 19. An aerial view of the Santa Clara River after construction of the levee is shown in figure 20. The placement of levee stone and toe stone is shown in figure 21.

On the Santa Maria River single and double levees were built to protect the area in the vicinity of the city of Santa Maria, from flood flows having subcritical velocities of 10 feet per second. The streambed material was medium and fine sand and silt. The design discharge was 150,000 c.f.s. and the maximum depth of flow would be about 8 feet. In this case no heavy toe stone was provided, but the depth of revetment stone was carried 10 to 12 feet below streambed because of the curvature in alinement. An aerial view of the Santa Maria River after construction of the single levee along the upper part of the project is shown in figure 22. Photographs showing the placing of the dumped stone revetment is given in figure 23. The stone had a maximum size of 300 pounds and was graded as follows:

Stone size:	*Percent smaller by weight*
300-pound	100
150-pound	60 to 90
75-pound	40 to 65
20-pound	10 to 30
5-pound	0 to 5

The design details are shown on figure 24.

FIGURE 21. — Riprap revetment on Santa Clara River levee: *A*, Downstream view showing filter course and facing stone; *B*, upstream view showing placing of heavy stone in toe trench.

Discussion and Conclusion

In order to place the revetment stone well below the streambed to protect against underscour, it is necessary to excavate a large toe trench. In practice, the excavation is usually done by dragline and the material cast onto a stockpile in the riverbed parallel to the levee. This material is later used for backfill in the toe trench and is usually not compacted. Thus, the streambed material adjacent to and over the levee toe where the greatest resistance is needed becomes the least resistant to erosion because it has been disturbed. It appears that where the streambed material is already fairly resistant to erosion, the point of diminishing returns may be reached in a deep toe excavation. Thus, judgment and experience appear to govern the design criteria rather than a theoretical approach to the solution of the question of how deep should be the levee toe.

In order to obtain field data on relative scour along a confining levee after construction, scour gages were devised to show maximum depth of scour during the peak of a flood, even though some filling might result on the recession side of the flood hydrograph. These scour gages consist of 15-foot deep holes, 12 inches in diameter, filled with colored stone granules. The horizontal and vertical location of the holes in a typical installation is shown in figure 25. Monuments were provided for location of the holes. The holes were drilled with a rotary-type auger drill. Wash water was not permitted. After drilling, the casing was removed as the stone granules were placed in the hole. The stone granules were specified as "Flamingo-Red natural stone (Roofing Granule) * * * reason-

ably well graded within the following limits: Percent by weight, passing square mesh sieve — 1½ inch size, 100 percent, ⅜ inch size, 0 percent." The magnitude of maximum scour during a flood will be determined by measuring after the flood the difference in elevation of the top of the stone granules remaining in the holes before and after the flood.

A board of consultants comprised of H. A. Einstein, L. G. Straub, and V. A. Vanoni was retained by the Los Angeles District of the Corps of Engineers to advise on criteria to be used in design of riprap levee revetment. Although the consultants were concerned primarily with details of design in each specific instance, some of the conclusions might be generalized: To obtain maximum effectiveness, riprap should consist of a dense mass of well graded, well interlocked angular stone rather than large individual particles. A thick layer of well graded small stone is better than a thin layer of large stone having correspondingly

FIGURE 22. — Upstream of Santa Maria River in Santa Barbara County, Calif., showing partial completion of the project. Right and left bank levees were subsequently built downstream from U.S. Highway 101 to contain 150,000 c.f.s.

FIGURE 23. — Downstream views of levee construction on Santa Maria River: *A*, General view showing toe trench excavation; *B*, closeup view of riprap revetment.

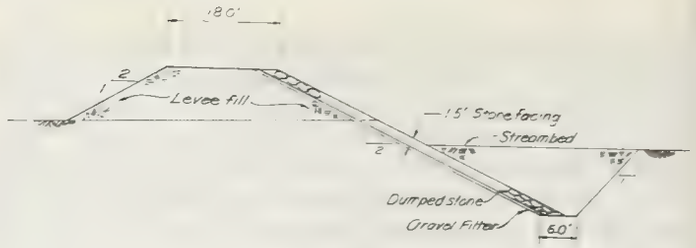


FIGURE 24. — Santa Maria River typical levee section.

large voids. Pressure fluctuations acting in the voids can lift large particles out of place and allow the single piece to be carried away. Curvature in the levee alinement induces attack along the toe on the outside of the curve. The radius of the curve should be made as long as practicable even at the sacrifice of expensive right-of-way. The toe of the levee revetment should be 6 to 12 feet below the thalweg of the stream, depending on character of material and hydraulic characteristics of the streamflow. If a meander provides a berm of 200-ft. width of undisturbed bank material between stream and levee, the depth of the toe may be reduced.

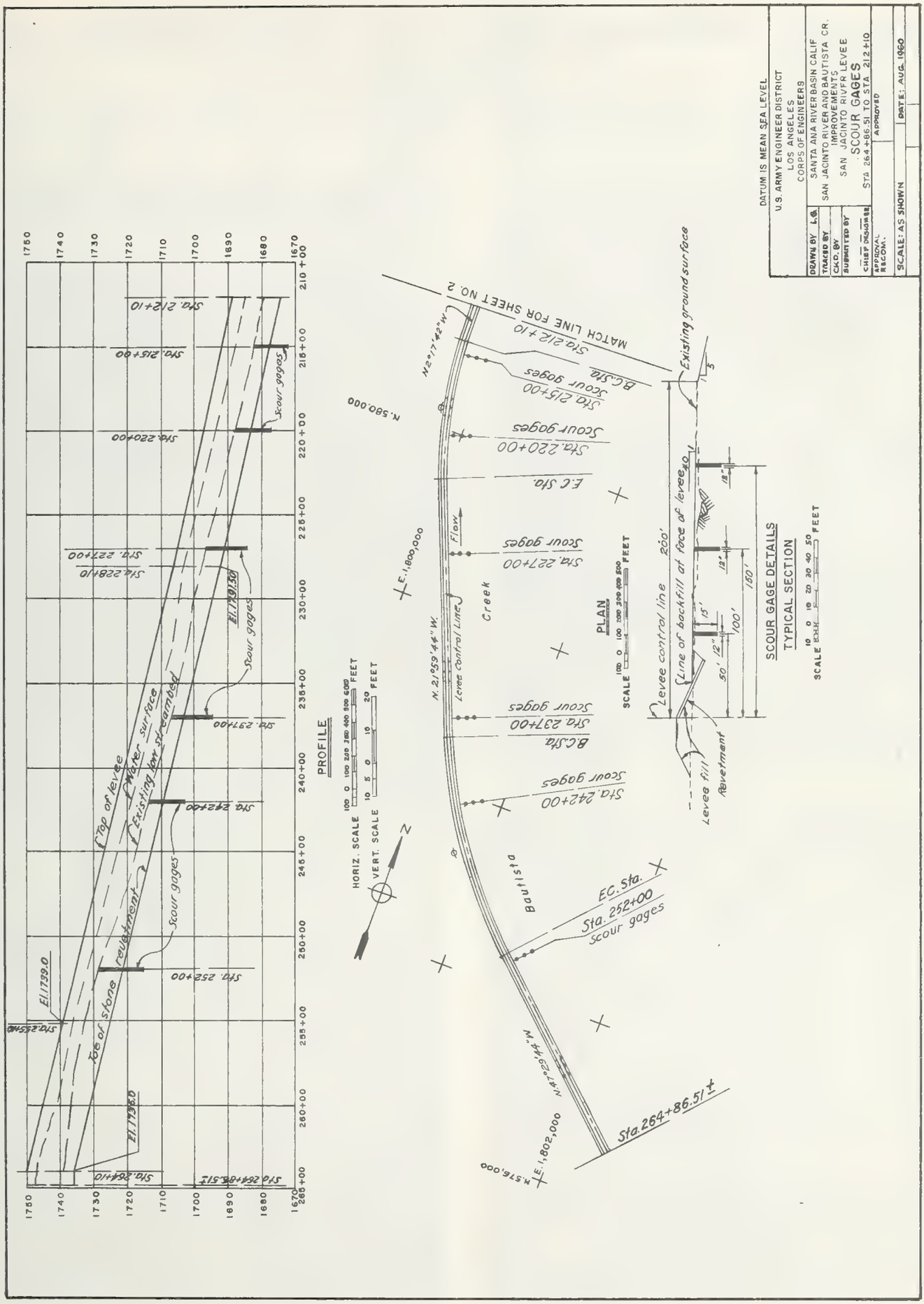


FIGURE 25. — San Jacinto River levee scour gages.

HYDRAULIC MODELS FOR STUDY OF RIVER SEDIMENTATION PROBLEMS

[Paper No. 58]

By JOHN J. FRANCO, *chief, Waterways Branch, Hydraulics Division, U.S. Army Engineers Waterways Experiment Station, Vicksburg*

Abstract

The paper reviews the state of knowledge concerning the hydraulics of natural streams and related sedimentation problems. It points out the difficulty encountered by the river engineer in the solution of sedimentation problems by analytical means and the reasons why the development of basic law and principles from experience in the field has not been practical.

The opinion is expressed that the advancement of the principles involved in river engineering and the solution of many sedimentation problems will have to depend for the most part on laboratory studies correlated with results of field investigations. The types of studies required are mentioned and the types of models used for these studies are described. One section of the paper is devoted to the design, verification and operation of the movable-bed model.

Movable-bed model studies in connection with the canalization of the Arkansas River and some of the results which could be applicable to alluvial rivers in general are discussed. The paper also describes an investigation in progress to determine the effects of water temperature on the movement of bed material and bed formations and includes preliminary results indicated by the study so far.

Developments within natural streams involving sedimentation are affected by many factors, some of which are interrelated and extremely complex. The struggle toward the regulation, stabilization, and improvement of natural streams for water supply, soil conservation, navigation, and flood control has been going on for centuries in all parts of the world. In spite of the vast effort and experience, few basic principles have been developed which can be used by the engineer in the design of river structures, and little is known as to why some structures are effective and some are not. The evaluation of conditions in the field is extremely difficult, because of the variation in geometry from one reach to the next, the variation in flow conditions, and the impracticability of comparing the effectiveness of various types and layouts of structures under the same conditions. Efforts have been made to obtain data on the performance of certain structures in the field, but very little of the available data have been evaluated and documented. In most cases, the data obtained have not been in sufficient detail to permit a reasonable evaluation of conditions before and after the installation of the improvement works.

Investigations of Sediment Problems

The study of the mechanics of sedimentation has been the object of many research projects. Much information is available covering the works of various experimenters, and the evaluations of the results of such studies have led to the development of several sedimentation formulas. Since these formulas were derived mostly from flume studies involving two-dimensional flow, they can be applied only in a general way to the solution of problems in natural streams. There is need for more research involving the factors affecting the movement of sediment, but the ultimate solutions of many practical problems will require research in the field and research with models of natural streams reproducing three-dimensional flow.

Hydraulic models have led to the development of some of the basic factors affecting the movement of sediment and the meandering of alluvial streams. Model studies have also pointed out some of the characteristics of structures affecting their performance in alluvial streams and have led to a better understanding of some of the processes involved in the development of channel configurations. However, the knowledge of the hydraulics of natural streams and of the sedimentation processes is not sufficient to permit the solution of many problems by analytical means with any degree of confidence. It is believed that the advancement of the science of river engineering, the optimum solution to many sedimentation problems, and the answers to many questions regarding the performance and effectiveness of various types and arrangements of structures will have to be based for the most part on the results of model studies correlated with results of field investigations.

Field investigations to be of value in the solution of sedimentation problems should be planned for that purpose and should be one of the major objectives of the project. When construction is involved, the investigation should be started in sufficient time to permit an evaluation of developments in the reach with low and high flows and should be continued during and after construction until the effects of the works involved have been definitely established.

Hydraulic Models

Hydraulic models used for the study of sedimentation problems are either of the fixed-bed type or of the movable-bed type. The study of problems in the fixed-bed type of model is based on the direction of movement of lightweight, plastic material or dye and current direction and velocity measurements. This type of study has certain advantages in that such models can be designed to obtain hydraulic similitude, are easier to adjust and less expensive to construct and operate, and results can be obtained within a comparatively short time. However, fixed-bed models are generally suitable only for the study of local problems in relatively stable streams involving the movement of small quantities of sediment.

The model generally used for the study of problems in alluvial streams is of the movable-bed type having fixed banks and overbank with the bed molded of a granular material such as sand, crushed coal, granulated plastic, etc. Erodible banks have been used to reproduce sections of banks where active caving is present in the prototype stream or where bank caving can be expected from the installation of the improvement plans to be tested; however, the use of erodible banks is usually not considered practicable because of the variations in the erodibility and other characteristics of the materials forming the banks of the natural stream and the difficulty of reproducing the conditions in the model with any degree of accuracy even if enough prototype data were available. When bank caving is a factor in the development of the stream, its effect can usually be simulated artificially in the model without compromising the ultimate results.

Movable-Bed Models

Design

The design of a movable-bed model must consider the characteristics of the stream involved, the problem or problems to be investigated, and the requirement that the material forming the channel bed be moved in simulation of the sediment movement in the prototype. Other factors that also affect the design of the model are space and facilities available and cost and time limitations. The scales selected for movable-bed models are generally too small to provide hydraulic forces sufficient to move material of a practical size and specific weight. The forces required to obtain general movement of the bed material are obtained by distortion of the linear scales or by use of supplementary slope. Distortion that involves the use of a vertical scale larger than the horizontal scale provides greater model depths and slope. Supplementary slope provides a tilt to the model in addition to that resulting from the linear scales, thus increasing the total model slope by a fixed amount. Distortion of the linear scales and supplementary slope tend to affect the relationship of velocity, width-depth ratio of the channel and curvature, and, consequently, the distribution of energy within the channel. Also, distortion produces changes in the size and shape, which in turn affect the performance of certain types of structures. Because of these effects, distortion and supplementary slope should be maintained as small as feasible. The effect of distortion on the performance of some structures such as sills or stilling basins can usually be eliminated by using the vertical scale to reproduce certain horizontal dimensions, particularly the dimensions in the direction of flow.

Verification

Complete similarity between a movable-bed model and its prototype can seldom be obtained, because of inherent distortions that are incorporated during model design and the impracticability of reducing the grain size of the bed material to the scale of the model. Because of the dissimilarities mentioned, the degree of similitude attainable in these models cannot be established by mathematical analysis. The reliability of a movable-bed model is usually based on the model verification. The normal verification is an intricate process of adjusting the various hydraulic forces and model operating technique until the model demonstrates its ability to reproduce with acceptable accuracy the changes in bed formations that are known to have occurred in the prototype during a given period.

The successful verification of the model is used to establish the time and discharge scales, the rate of introducing bed material for each flow reproduced, model operating technique, and the degree of similitude that can be expected. The verification involves considerable time and effort, careful observation and analysis of developments, and the exercise of engineering judgment in the evaluation of the factors affecting development in the prototype and model. The degree of similitude attainable and the time required for model verification depend upon the amount and type of prototype data available, the scales of the model, experience with models of this type, and the refinements to which the adjustment of the model is carried.

Test Procedure

After the model has demonstrated its ability to reproduce conditions similar to those that occurred in the prototype and before the tests of various plans are undertaken, a "base test" is conducted. For this test the model is usually operated by reproducing typical, or average,

annual stage and discharge hydrographs until developments within the model channel resulting from the flow conditions reproduced have become stable. Improvement plans are tested with the same hydrograph used in the base test, and the results of the latter form the basis for determining the relative effectiveness of the various plans. However, under certain conditions, tests might also be conducted by reproducing a hydrograph including an unusually low-water or high-water period.

Models Used in Study of Arkansas River Problems

Hydraulic models of the movable-bed type have been concerned mostly with the investigation of specific problems, and the scopes of these studies generally have not been sufficiently broad to permit the development of basic information applicable to streams in general. Some of the more versatile types of movable-bed model studies have been concerned with problems in connection with the canalization of the Arkansas River. Studies completed or in progress are concerned with such problems as design and location of lock and dam structures, effects of the structures on the movement of sediment, development of channel depths and alinement, radii of curvature of bends, effectiveness of various types and arrangements of regulating structures, and navigation conditions in the approaches to the locks.

The models used for the study of problems in the Arkansas River have been constructed to fairly large scales to permit studies to be made in greater detail and to facilitate the study of the varied problems involved. The horizontal scales used in these models have ranged from 1:120 to 1:250, with vertical scales ranging from 1:36 to 1:80; distortion of the linear scales (vertical scale divided by the horizontal scale), has ranged from 1.5 to 4.17. The time scales developed for these models were such that one day of flow in the prototype was reproduced in from 5 to 10 minutes; the reproduction of an annual hydrograph would therefore require from 30.4 to 60.8 hours of operation. The test of a modification usually requires several reproductions of the annual hydrograph, depending upon the extent of change produced by the modification, before definite indications are established. Since the time required to test a given plan is appreciable and the time available before the information is usually needed by the design engineer is limited, these studies of necessity have to be confined generally to the development of a practical solution rather than to the development of a refined optimum solution.

The results of studies in connection with the canalization of the Arkansas River have brought out some information that should be applicable to the regulation and improvement of natural streams in general. These studies have shown how the use of spur dikes, particularly on the concave side of a bend, can produce considerable disturbance to flow, reduce channel efficiency (increase roughness), and could produce shoaling upstream and downstream. It has been demonstrated that dredging of the channel bed without other changes would produce only a temporary effect, regardless of the size of the cut. The studies have emphasized the importance of the lower reach of a bendway on developments downstream. The depth and alinement of the channel over a crossing can be improved, and the depth of scour and current attack along the revetted bank in a bend can be reduced by proper design of the lower reach of the bend upstream.

Effect of Water Temperature

It has long been noted that the movement of bed material in movable-bed models can be affected by the temperature of the water. With materials of a certain grain size, riffles would form with cold water and not form with water at a higher temperature. It was also noted that the temperature at which a given material would riffle could be lowered by increasing the median-grain size of the material. It has been suspected from the above that the differences in the stage-discharge relation observed in alluvial streams might be caused by the difference in the temperature of the water. Research investigations are in progress to determine the effect of water temperature on the movement of bed material, bed formations, and channel efficiency.

This research is being conducted in a tilting flume equipped with a water circulating system, water temperature control, automatic bed-material feeder and a device that automatically records the weight of the material extruded. Tests are being conducted with a constant discharge, rate of bed feed, and tailwater elevation for each of three water temperatures (40°, 60°, and 80° F.). The flume is operated until the rate of material extruded and the water-surface slope have become stabilized for each test condition. The first series of tests has been conducted with a fairly uniform sand having a median grain diameter of about 0.2 mm. A considerable amount of data was obtained during these tests, but analysis of the results has not progressed sufficiently to provide definite conclusions. Preliminary plots of some of the data obtained show an increase in the energy gradient with a decrease in the water temperature for the same rate of bed movement (fig. 1). This could be an indication that the carrying

capacity of the water decreases with temperature or that the formations (riffles) along the bed are changed so as to increase the roughness of the bed as the temperature of the water is lowered (fig. 2).

The material used in the first series of tests riffled at all three of the water temperatures tested. It is planned to conduct the next series of tests with a material that does not riffle at those temperatures. A comparison of the results of the two series of tests might provide some of the answers.

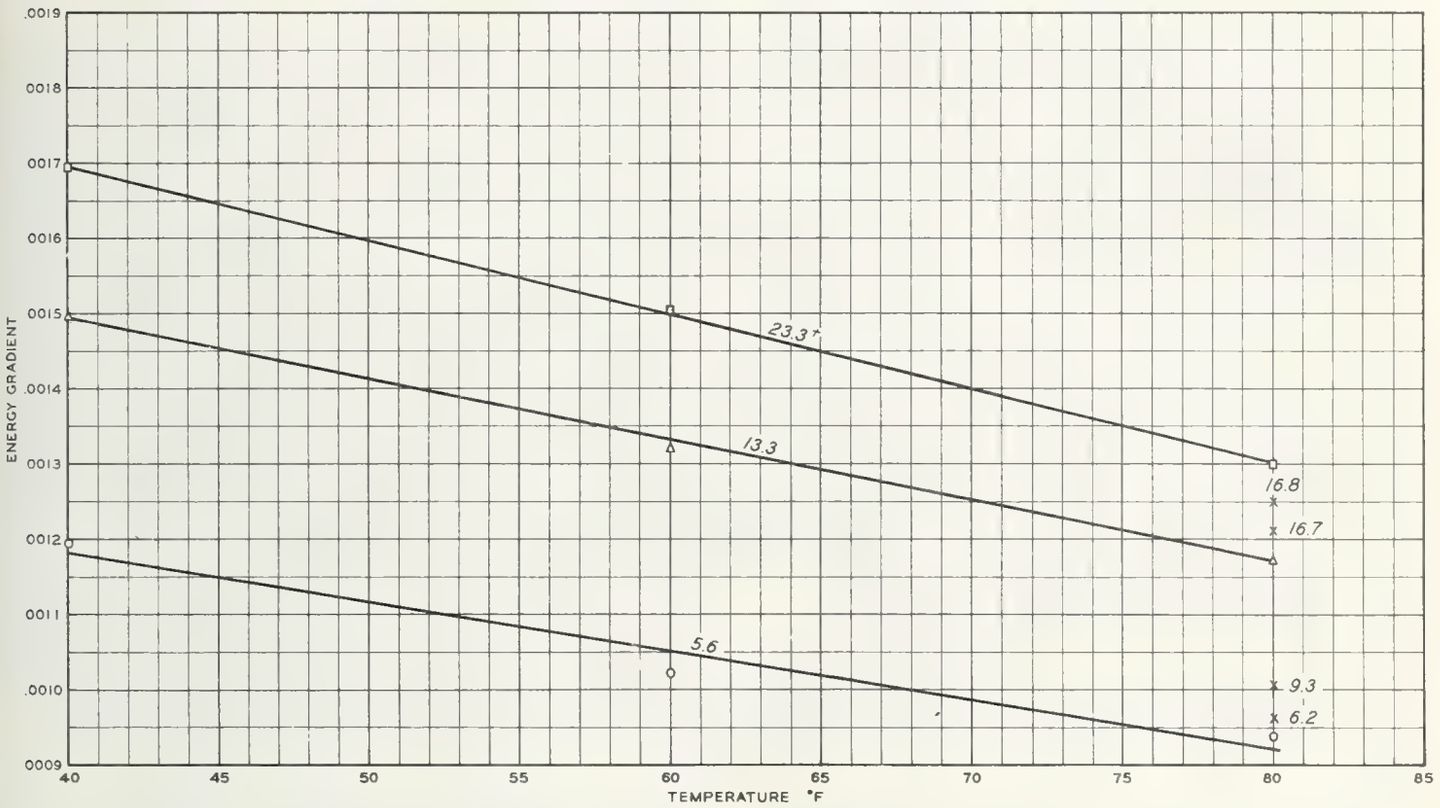


FIGURE 1. — Energy gradient-temperature curves; fine sand bed material.

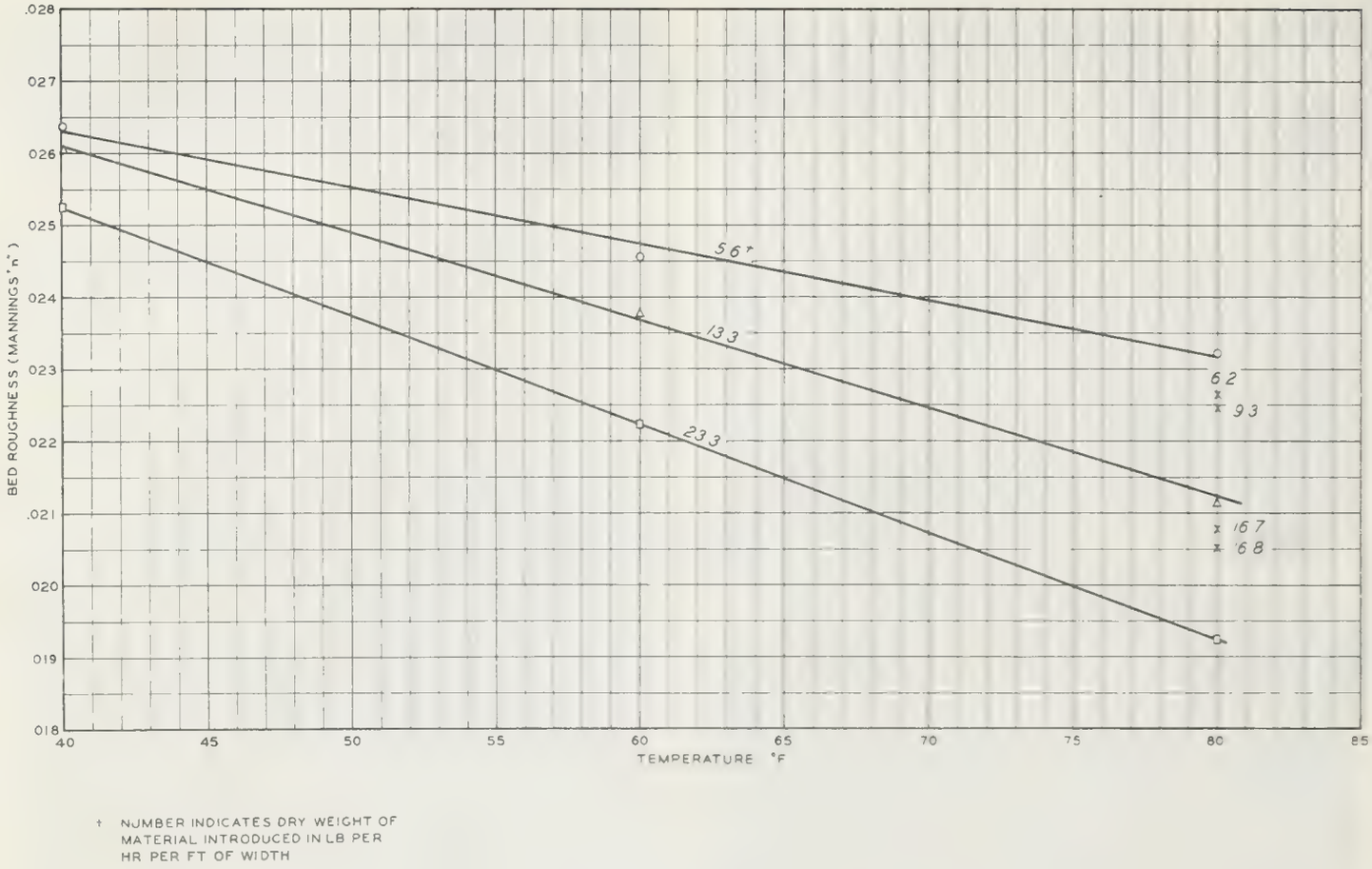


FIGURE 2. — Bed roughness-temperature curves; fine sand bed material.

LOWER HUDSON RIVER SILTATION

[Paper No. 59]

By FRANK L. PANUZIO, *assistant chief, Engineering Division for Civil Works, U.S. Army Engineer District, New York*

Synopsis

Siltation of the navigable channel and adjacent slips of the Lower Hudson River, a tidal estuary of the Port of New York, is a major item of local and Federal maintenance and a deterrent to the full use of the river frontage for maritime purposes.

To obtain a solution, analysis of field prototype data is being augmented by studies on two complementary models that integrate the nonanalytical complex factors and influences involved in the hydraulic circulation of fresh and salt water and in the siltation processes of this tidal estuary. A comprehensive model to study hydraulic circulation and channel shoaling reproduces 100 miles of the New York Harbor and the Hudson River from the Atlantic Ocean to Poughkeepsie, N. Y. An enlarged sectional model to study slip shoaling reproduces 9 miles of the Lower Hudson River from 1 mile upstream of the Battery to 1 mile downstream of George Washington Bridge.

Fresh and salt water circulation and siltation processes within this estuary; variation of shoaling quantities and pattern before and after a change in geometry at a critical point of the estuary; cross sectional distribution of velocity, salinity, and suspended sediment flow; and comparative model results of possible solutions involving sediment traps, dikes and barriers, and changes affecting channel geometry or hydraulic circulation and salinity are discussed.

The final solution to the problem must await the results of further model tests and final evaluation of hydraulic and economic factors.

Introduction

The lower Hudson River is a principal

navigable estuary of the Port of New York and the entrance to the Hudson River channel to Albany, N. Y. Heavy shoaling is occurring in the 11 miles of Federal channel and in 250 adjacent slips. In addition to the Federal cost of annually dredging about 1,600,000 cubic yards of sediment from the navigable channel, local interests incur the annual cost of removing about 3,000,000 cubic yards of sediment from the slips. Because of this heavy local cost, full use is not being made of the river frontage for maritime purposes. So, in July 1959, at the request of local interests, the Public Works Committee of the House of Representatives authorized a study, currently underway, to determine what improvements are necessary and feasible to lessen shoaling in the pier slips and in the Federal channel.

Even though this shoaling problem is unusually severe, it is typical in that all partially mixed estuaries have similar shoaling problems where fresh water runoff contacts salt water, causing flocculation and settlement of fine materials in suspension. The problem is complex because the interplay of fluvial and tidal currents in the lower Hudson River defy mathematical analysis, resolution, and expression. To solve the problem, field measurements and studies are being supplemented by measurements and studies on two hydraulic models that integrate effectively the many forces, factors, and influences involved in the problem.

The Hudson River

The roughly cross-shaped watershed of the Hudson River (fig. 1) extends 120 miles east to west and 236 miles north to south. This basin (fig. 2) drains 13,400 square miles of the northern and eastern parts of New York with small areas in Vermont, Massachusetts, Connecticut, and New Jersey. From its source in the Adirondack Mountains, the river flows generally south to its junction with its principal tributary, the Mohawk River. The main river continues to flow in a southerly direction for 155 miles to its mouth at the Battery, where it empties into the Upper New York Bay.

In its lower reach (fig. 2), the Hudson River has been improved for 11 miles for navigation, with depths varying from 30 to 48 feet below mean low water from the Battery to a point downstream of George Washington Bridge. Upstream of this point, for 143 miles to Albany, the navigable depth is 27 feet and work is underway to deepen it to 32 feet below mean low water. Upstream of Albany the rivers have been canalized as a part of the New York State Barge Canal System. North of Albany 49 miles of the Hudson River have been canalized with depths of 12 to 14 feet to connect with the nar-

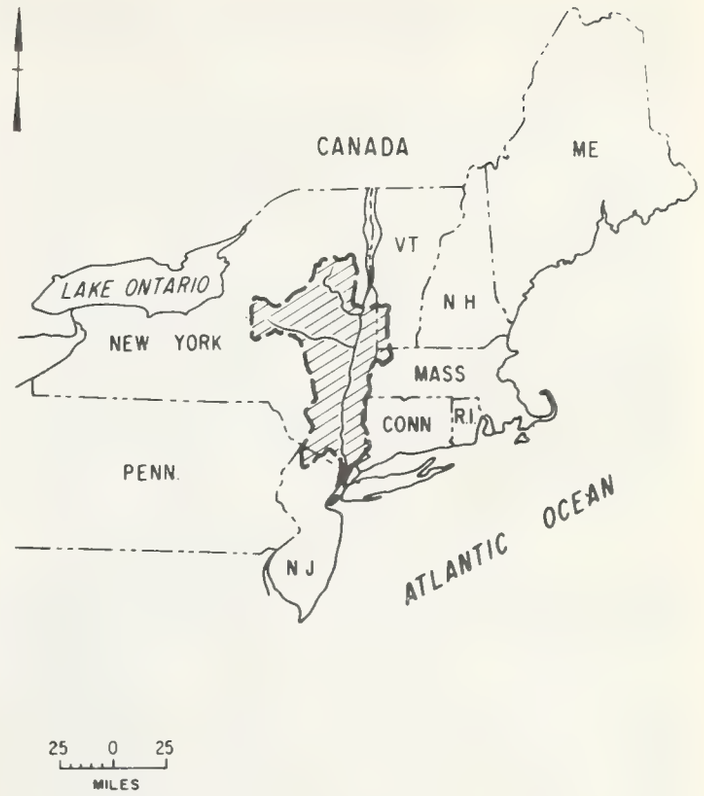


FIGURE 1. — Watershed of Hudson River.

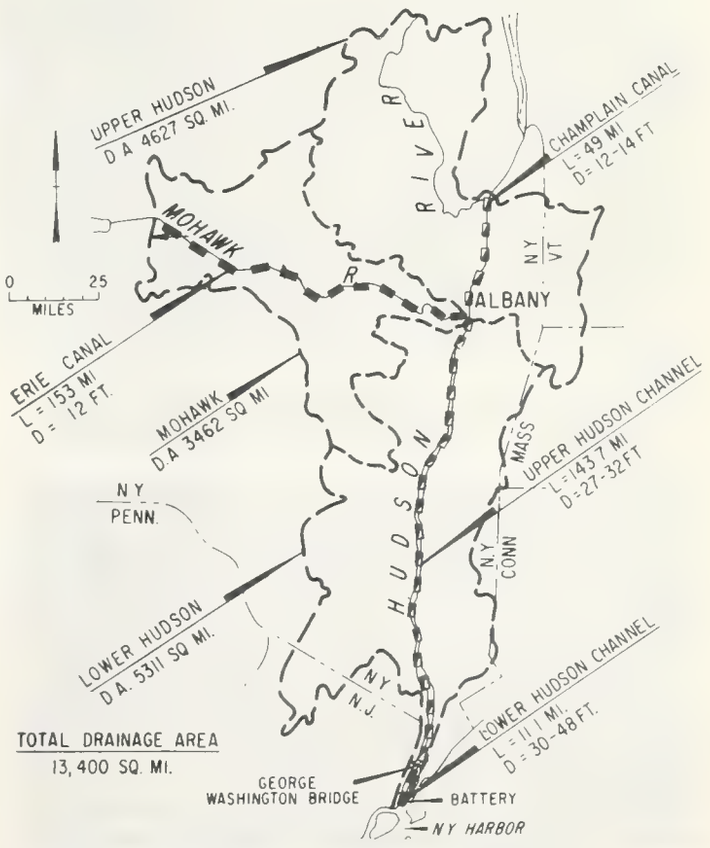


FIGURE 2. — Characteristics of Hudson River basin.

rows of Lake Champlain. West of Albany, 153 miles of the Mohawk River have been canalized to a depth of 14 feet to connect with Lakes Erie and Ontario.

The average annual precipitation over the basin is 43 inches, of which 24 inches appears as annual runoff at the mouth to produce an average fresh water runoff of 23,000 cubic feet per second.

The tidewater in the Hudson River extends 154 miles from its mouth at the Battery to the Federal Lock and Dam at Troy, 10 miles upstream of Albany, N.Y. The mean tidal range decreased from 4.4 feet at the Battery to a minimum of 3.0 feet at Beacon, N.Y., 62 miles upstream of the Battery, and then increases to 4.8 feet at Troy.

Basic Hydraulic Mechanism for Sedimentation

The New York Harbor, from the viewpoint of circulation, represents probably the most complex major estuarine system in the United States. The principal hydraulic components (fig. 3) are Lower New York Bay, Sandy Hook Bay, Raritan Bay, Newark Bay, Arthur Kill, Kill Van Kull, Hudson River, East River, Harlem River, the lower reaches of Hackensack and Passaic Rivers, the Atlantic Ocean, and Long Island Sound.

Most of the fresh water entering the system is contributed by the Hudson River (13,400 sq. mi.) with some contribution from the Raritan, Hackensack, and Passaic Rivers (2,300 sq. mi.) that for short periods may be significant. The tides in the principal components are controlled primarily by the ocean tide entering directly into the Lower Bay, and indirectly through the Long Island Sound into the East River (fig. 3). The degree of mixing that influences the circulation pattern varies from highly stratified to well mixed, depending on the magnitude of the tidal flows as compared to the fresh waterflows.

Studies of the New York Harbor in nature and models revealed many valuable aspects of circulation.[1]

The plan of circulation in the New York Harbor is extremely complicated. The Hudson River, upstream of the Battery, is a system in which the inertia of a long column of moving water must be overcome by the rising or falling

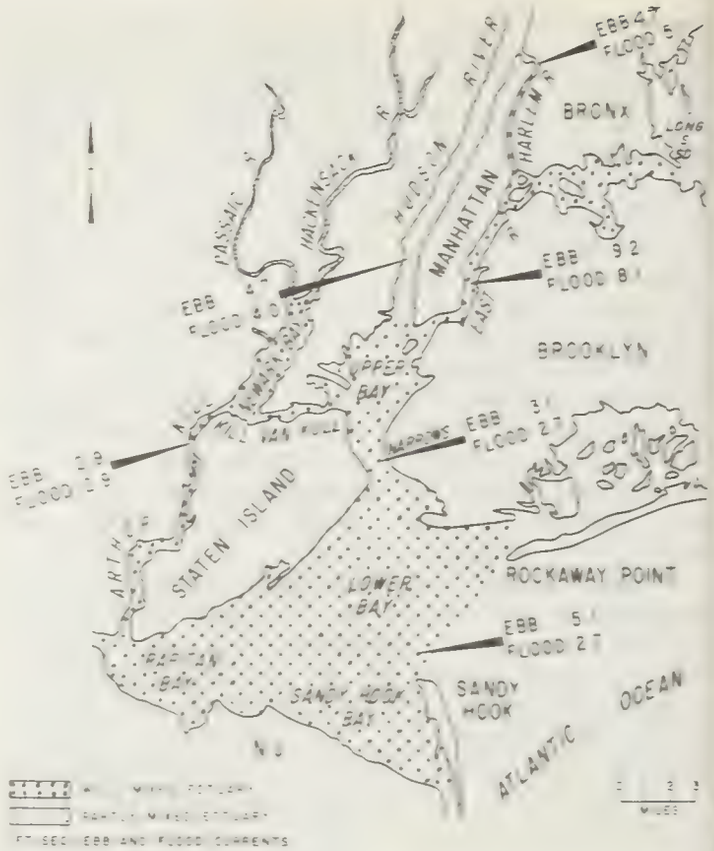


FIGURE 3. — New York Harbor estuarine system.

tide head before slack current can be effected. This is best demonstrated by the current pattern 2 hours after low water at the Battery. At this time (fig. 3) the Hudson River is ebbing at the surface and receiving flood discharge of the Upper New York Bay at the bottom; the East River is receiving ebb discharge of the Hudson River at the surface and flood discharge of the Upper New York Bay at the bottom; Newark Bay is receiving ebb discharge of Hudson River and flood discharge of the Upper New York Bay, and the upper New York Bay is receiving flood discharges of the Lower Bay through the Narrows.

This difference in phasing of currents to some extent affects the sedimentation pattern in the Lower Hudson River. However, it has been found from model observations of the basic flow mechanism operating in the Harbor, that the Hudson River is to some extent insulated from the rest of New York Harbor as far as receiving sediments is concerned. The fresh waterflow of the Hudson River constitutes a sufficiently large percentage of its total flow to be important in the establishment of density currents. However, in the Lower Bay, Raritan and Sandy Hook Bays, and the Upper New York Bay, the fresh water discharges represent a small percentage of the total tidal discharge, so

[1] DUKE, CHARLES M. SHOALING IN THE LOWER HUDSON RIVER. Amer. Soc. Civil Engin. 2730. February 1961.

FORTSON, E. P. JR., and SIMMONS, HENRY B. SOME ASPECTS OF CIRCULATION REVEALED BY THE NEW YORK HARBOR MODEL. Amer. Soc. Civil Engin. meeting. 12 pp. October 1959.

NEW YORK HARBOR COMMITTEE ON TIDAL HYDRAULIC. REVIEW OF SHOALING PROBLEMS IN HUDSON RIVER. U.S. Army Corps of Engineers. 24 pages. July 1961.

that the water is well mixed vertically. Thus, the density currents in Upper New York Bay are relatively weaker than those in the Hudson River proper. Therefore, it is improbable, though not impossible, that any considerable source of shoaling to the Hudson River could be located below the Battery.

The difference in phasing of the currents in East River and Hudson River causes the ebb flow in the East River to flow upstream into the Hudson River for about 1½ hours before the Hudson reaches flood slack and begins to ebb. This could make it possible for the East River to be a source of shoaling material for the Hudson River.

The part of the Hudson River within the New York Harbor area from the Battery to about Haverstraw Bay is normally classified as a partly mixed estuary (figs. 3 and 4) Tidal currents go through a complete reversal from ebb

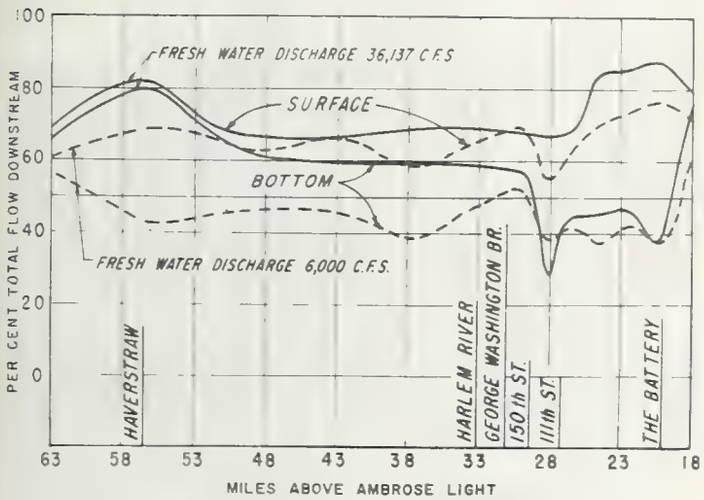


FIGURE 4. — Flow predominance at surface and bottom of Hudson River within the New York Harbor.

to flood at all depths throughout this reach, but vertical mixing is not complete. For favorable conditions of location, fresh water flow, and phase of tide, salinities at the bottom may be as much as 10 parts per thousand greater than at the surface (fig. 4). This large salinity gradient causes the reversal of the current from ebb to flood to occur by as much as 2 hours earlier at the bottom than at the surface, so that the strengths and durations of the bottom flood currents are greater than those of the bottom ebb currents. As a result, more water flows upstream than downstream at the bottom; and, conversely, more water flows downstream than upstream at the surface. In general, the salinity-induced upstream flow predominates over the downstream flow in about the lower half of the stream, and the fresh-water induced downstream flow predominates over the upstream flow in the upper half.

The magnitude and extent of the upstream predominant bottom currents vary with the amount of fresh water flow in the Hudson River (fig. 4). At high flows these currents are increased in magnitude but reduced in extent, so that the upstream limit is downstream from the George Washington Bridge. At medium and low fresh waterflows, they extend well upstream of the George Washington Bridge, but are interrupted locally just downstream of the bridge.

The interruption of the upstream predominant bottom currents during medium and low fresh waterflows in the Hudson River is due to a combination of two conditions that are considered to effect sedimentation significantly in the Lower Hudson River.

One condition is the abrupt change in cross-sectional area in the vicinity of the George Washington Bridge (fig. 5). The constricted

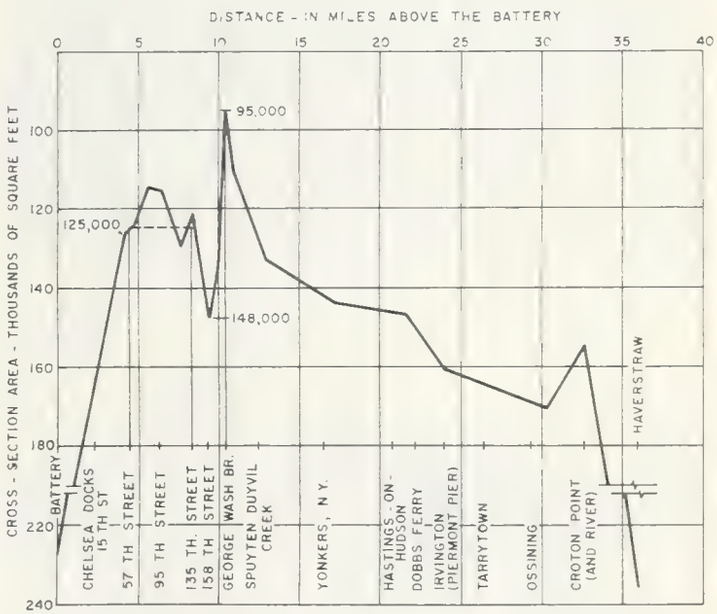


FIGURE 5. — Lower Hudson River: cross sectional areas at mean low water.

cross-sectional area near the bridge causes an intense turbulence that effectively interrupts the upstream predominant bottom currents but still permits the movement upstream of saline water that reforms the upstream predominant bottom currents upstream of the George Washington Bridge (fig. 4). The excessive cross-sectional area occurring downstream of the bridge where the bottom flood predominance is either enfeebled or eliminated, depending on the fresh water discharge, tends to receive sediment that otherwise might be induced to proceed farther upstream before depositing.

The other condition is that the discharge of large quantities of heavy saline water from the Harlem River into the Hudson River upstream of George Washington Bridge (figs. 3 and 4) during certain phases of the tide reduces the vertical salinity gradient and interferes with the upstream bottom predominance.

Thus, there are two separate and independent reaches in the Hudson River where upstream predominant bottom currents exist during medium and low flows, one upstream and the other downstream of the George Washington Bridge.

The movement of silt within the lower Hudson River is related to and controlled by the vertical distribution and pattern of currents as herein described. Thus, suspended silt entering the area is first carried downstream in suspension in the upper fresh water, predominantly downstream layers, then gradually dropped to the bottom layers through flocculation and settling processes. Material reaching the bottom layer gradually moves upstream rather than downstream until it is deposited in areas where the bottom current velocities are too low to move material or where the net movement of bottom currents is zero. More specifically, from the downstream end of the shoal areas (fig. 6) material is moved upstream along the navigation channels to West 59th Street (see fig. 20). From this point, the material moves upstream in the naturally deeper channel along the New York side.

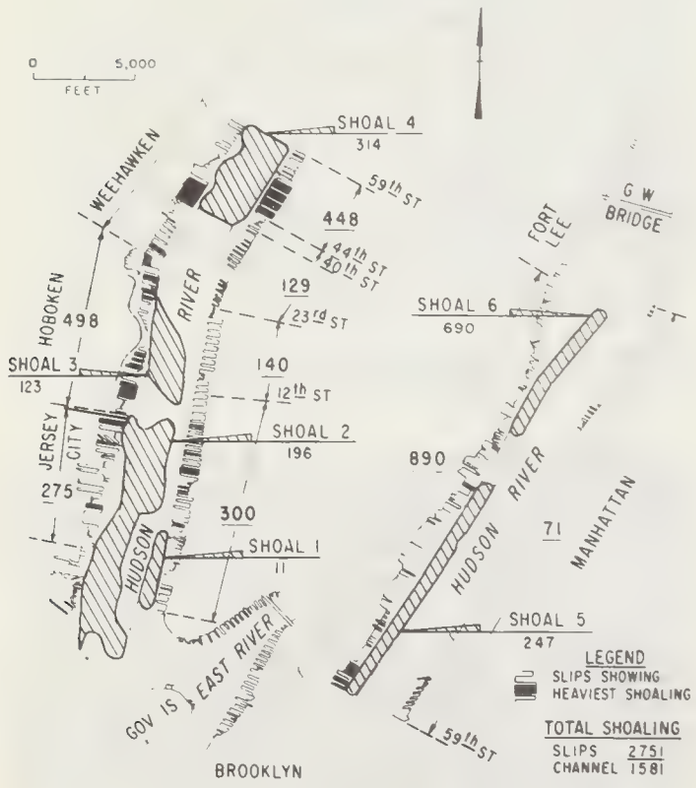


FIGURE 6. — Slip and channel shoaling (thousands of cubic yards), Hudson River.

As heretofore noted, the upstream predominance varies with depth and, in the region from 59th Street to George Washington Bridge, is confined to the east (New York) side of the Hudson with natural depths of 40 to 50 feet as compared to 30-foot depth on the west (New Jersey) side. When the material reaches the George Washington Bridge, the turbulence throws the material into the surface layers and toward the New Jersey side. Then the material is deposited into shoal areas by ebb currents moving downstream in the 30-foot channel on the New Jersey side in which there is no predominant upstream current.

More simply, a significant part of the shoal material in the Lower Hudson River moves intermittently but progressively in a counterclockwise direction, generally upstream along the New York side to the vicinity of George Washington Bridge, then downstream along the New Jersey side.

Some material is carried upstream on floodtides through the George Washington Bridge, after which it reenters the upstream predominant bottom currents of low and medium freshwater flows and moves in naturally deep water upstream of the bridge until deposited in the shoals in vicinity of Haverstraw Bay, which is also fed by upstream materials.

The shoaling of the slips is related to this sedimentation mechanism that is responsible for the presence of shoaling materials that permeate the waters in the lower strata of the harbor. The shoaling of the slips is due to a combination of causes. The current velocities associated with filling and emptying the individual tidal prisms of the slips are very slight, but the settling rate of the suspended load is slow and small velocities are competent to carry the material. The flooding of the slips is effected principally from the bottom currents of the main stream, the greater part of it entering in the first 2 to 3 feet above the bottom. Therefore, not only is the velocity greater because it occupies a small cross section, but it occurs at a level at which the concentration of suspended load is greatest, so that large amounts of sediment are carried into slips on every floodtide. Conversely, the emptying of the slip prisms involves approximately equal flow throughout the slip cross section, and the outward moving bottom velocities from the slips during ebbtides do not have sufficient velocity to remove the silt that has come into the slips during floodtide. The net result is that silt enters the slips during each floodtide and very little is moved out on the following ebbtide.

Nature and Extent of Sedimentation

In order to understand better the factors entering into the sedimentation processes and to obtain data required to construct and to operate a model that would reproduce the basic natural sedimentation mechanism, studies of past records and measurements were made of shoaling pattern and amounts, source of sediment, and vertical distribution of salinity, velocity, and sediment discharge.

Channel Shoaling

The distribution of channel shoaling was developed from maintenance surveys and dredging records. The annual amount of material distributed in some six shoal areas (fig. 6) amounts to about 1,600,000 cubic yards annually in the navigation channels. The heaviest shoaling occurs in the 30-foot channel, shoals 5 and 6, where over 900,000 cubic yards of materials have been dredged annually.

Slip Shoaling

The distribution of slip shoaling was similarly developed from available incomplete records extrapolated to cover the entire area. The annual amount of slip shoaling was determined as about 2,700,000 cubic yards, of which 60 percent occurs on the New Jersey side (fig. 6). The problem slips, noted in black on figure 6, are generally located adjacent to the channel shoal areas. However, an increase in slip shoaling also varies directly with an increase in depth of slips, 15 to 18 feet are required for barges and ferries, 26 to 28 feet for cargo vessels, and 40 feet and more for passenger vessels and supertankers.

Spoil Dumping in Harbor

Dumping of dredged material in the New York Harbor during World War II may have modified the shoaling pattern and rate in the Lower Hudson River. During this time, over 53 million cubic yards of mud, clay, sand, and rock were dumped at 11 or more deep water or nonnavigable locations in the New York Harbor area. Of this amount about 21 million cubic yards were dumped in the Narrows, over 13 million cubic yards under George Washington Bridge, and over 7 million cubic yards at Governors Island. Recent surveys indicate that only a small amount of the dumped material remains, except at George Washington Bridge where over 40 percent of the dumped material, believed to be rock, still remains.

The effect on shoaling due to the change in geometry at the George Washington Bridge has been appraised, based on studies of maintenance dredging records, corrected for new work, before and after the dumping operation. Fifteen years of maintenance records before dumping (1926-40) and after dumping (1946-60) were corrected and plotted as mass curves (fig. 7).

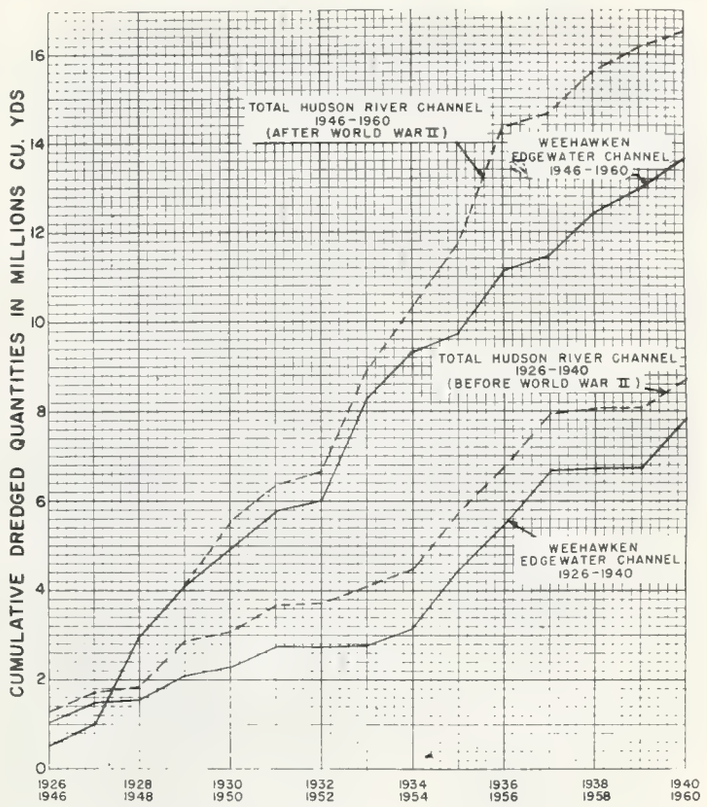


FIGURE 7.—Mass curve of dredged quantities before and after World War II spoil operations, New York Harbor.

Between 1926 and 1940, an average of 500,000 cubic yards were dredged annually for maintenance, of which 490,000 cubic yards represent the quantity dredged from the Weehawken-Edgewater channel and 90,000 cubic yards from the rest of the project. During 1946-60, the 15 years after dumping in the harbor, an average of 1,130,000 cubic yards were dredged annually for maintenance, of which about 930,000 cubic yards were in the Weehawken-Edgewater channel and 200,000 cubic yards in the rest of the project. These figures represent an increase in shoaling of about 100 percent.

Source of Shoaling Material

Even though, as previously described, the hydraulics of the harbor system results in a mechanism for moving and depositing suspended material that is independent of the source of shoal material, limited measurements of suspended sediment and an accounting of other material entering the lower Hudson River were made. The probable sources of shoaling material considered were the Hudson River watershed; local

dumping through sanitary and storm sewers, industrial, and others; other watersheds entering New York Harbor, Long Island Sound, and the Atlantic Ocean.

Sediments from Hudson River Watershed

The Hudson River watershed furnished sediment from erosion of the land areas, streambeds, and streambanks (fig. 8). The material is

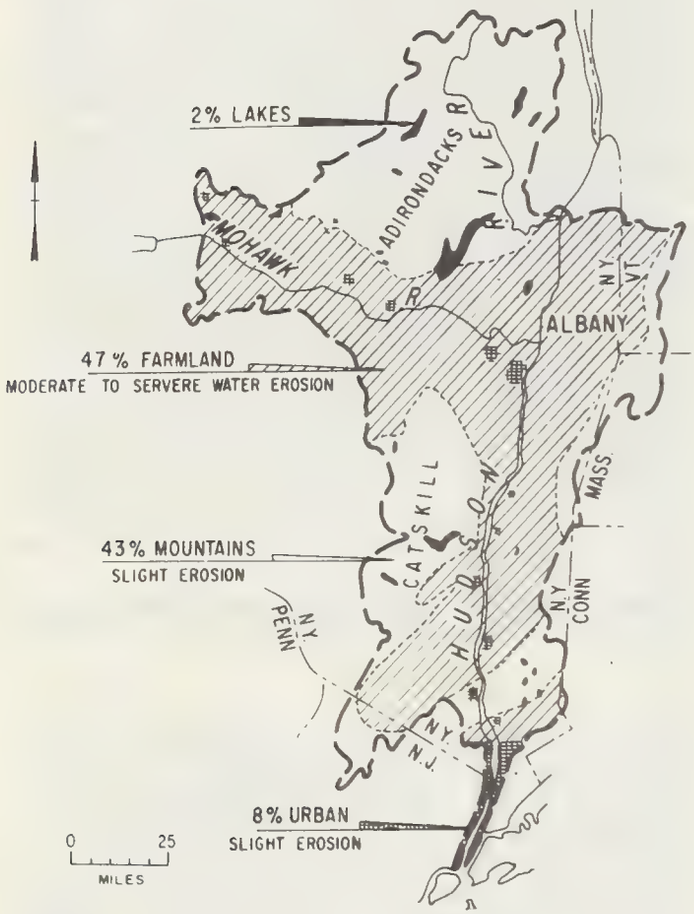


FIGURE 8. — Physiography and erosion of Hudson River watershed.

carried by the river in suspension or is moved by the current along the streambed. In this investigation, no attempt was made to measure the bedload. The suspended load was measured by means of daily water samples taken at Poughkeepsie, N. Y., which is 75 miles upstream from New York Harbor and above the limit of salt water intrusion. The Department of Public Works at Poughkeepsie cooperated in obtaining twice-daily water samples for the period 3 July 1959 through 15 September 1960. The sampling period encompassed the seasonal fluctuations of fresh water discharge and sediment concentration and provided a complete year's record of sediment discharge for comparison with the annual prototype shoaling volume in the problem area. Salinity as measured in the water samples at Poughkeepsie ranged from 8 to 35 p.p.m., averaging 15 p.p.m. for the test period.

The structure for the Poughkeepsie water intake is located at the midpoint of the river, which is about 2,000 feet wide, at depth of 54 feet below mean low water, or 5 feet above the deepest part of the cross section. Water is drawn through the manifold into a 36-inch pipe leading to a pumphouse. To insure a representative sample, a continuous discharge was maintained through an open tap where quart samples were taken and sealed in mason jars. Duplicate quart samples were obtained twice a day; one set at strength of ebb and the other at strength of flood for the period 1 September 1959 through 31 August 1960. Additional bimonthly samples were taken for chemical and mechanical analyses.

The samples of water were tested for total suspended solids by standard methods. The sediment concentration, the weight of dry solids to the weight of the sample, were recorded in parts per million. The total daily suspended sediment discharge was the product of the total fresh water discharge at Poughkeepsie in a day second-feet times the sediment concentration during ebb flow. The daily fresh waterflows were obtained from flow records of the Hudson River at Green Island, opposite Troy, N. Y., and the intermediate tributary streams, including Kinderhook, Wappinger, and Rondout Creeks. The time-interval required for fresh waterflows from Green Island, the primary gaging station, to reach Poughkeepsie was developed as 1 day. Therefore, in computing the suspended sediment discharge at Poughkeepsie from the upstream fresh waterflows a 1-day lag was applied. The suspended sediment concentration at strength of ebb current and the daily fresh waterflow at Poughkeepsie were used in computing the daily sediment load flowing to the problem area, because any sediment moving upstream during the flood cycle necessarily had to return during subsequent ebb cycles.

In order to determine the relation of the single sample intake concentration to the mean sediment concentration for the entire river cross section, sediment samples were taken at 10 various depths and locations in the cross section at the Poughkeepsie intake during the ebb flow. These samples were analyzed for sediment concentration and the results averaged 68.5 p.p.m. All the samples were taken within an hour of the two intake readings, which averaged 78 p.p.m. Based on these tests, the intake readings were found to be higher than average and a coefficient of 0.88 was derived to reduce the

intake recording to a mean suspended sediment concentration for the entire river cross section. A further adjustment was necessary, so that the sediment concentration taken at strength of ebb current could be converted to a mean for the ebb current duration equal to the ratio of maximum current 1.2 knots to mean current 0.4 knot for a coefficient of 0.33. Combining the two coefficients provided a constant of 0.3, which applied to the observed intake concentration in parts per million would provide a mean concentration representing the total suspended sediment passing Poughkeepsie each second. This study confirmed the fact that sediment concentration increases from the water surface to the bed of the river with average concentrations recorded at 0.6 depth and that distribution of suspended sediment varies in the cross section, being heaviest at the center and deeper parts of the river.

The daily sediment load, in tons passing Poughkeepsie and flowing to the lower Hudson River, was computed as follows:

Daily sediment load at Poughkeepsie (tons/day) $= Q$ (c.f.s.) $\times$ 86,400 (sec./day) $\times$ 62.4 (lb./cu. ft.) $\times \frac{1}{2{,}000}$ (ton/lb.) $\times \frac{\text{p.p.m.}}{10^6} \times 0.3$

$= 0.00808704 \times$ p.p.m. $\times Q$ (c.f.s.)

On the basis that the total suspended sediment transported past Poughkeepsie reaches the Lower Hudson River, the daily sediment load in tons was obtained by correcting sediment flow at Poughkeepsie by a factor equal to ratio of effective drainage areas of 1.1 (12,730 sq. mi./11,407 sq. mi.) or $0.008976614 \times$ p.p.m. $\times Q$ (c.f.s.).

The results of these measurements, more specifically the fresh water discharge at the problem area, average daily concentration of suspended sediment in parts per million through the ebb current at Poughkeepsie, daily suspended sediment load transported to the problem area for the period 4 July 1959 through 15 September 1960, and daily precipitation measured at Albany, N. Y., are shown on figure 9.

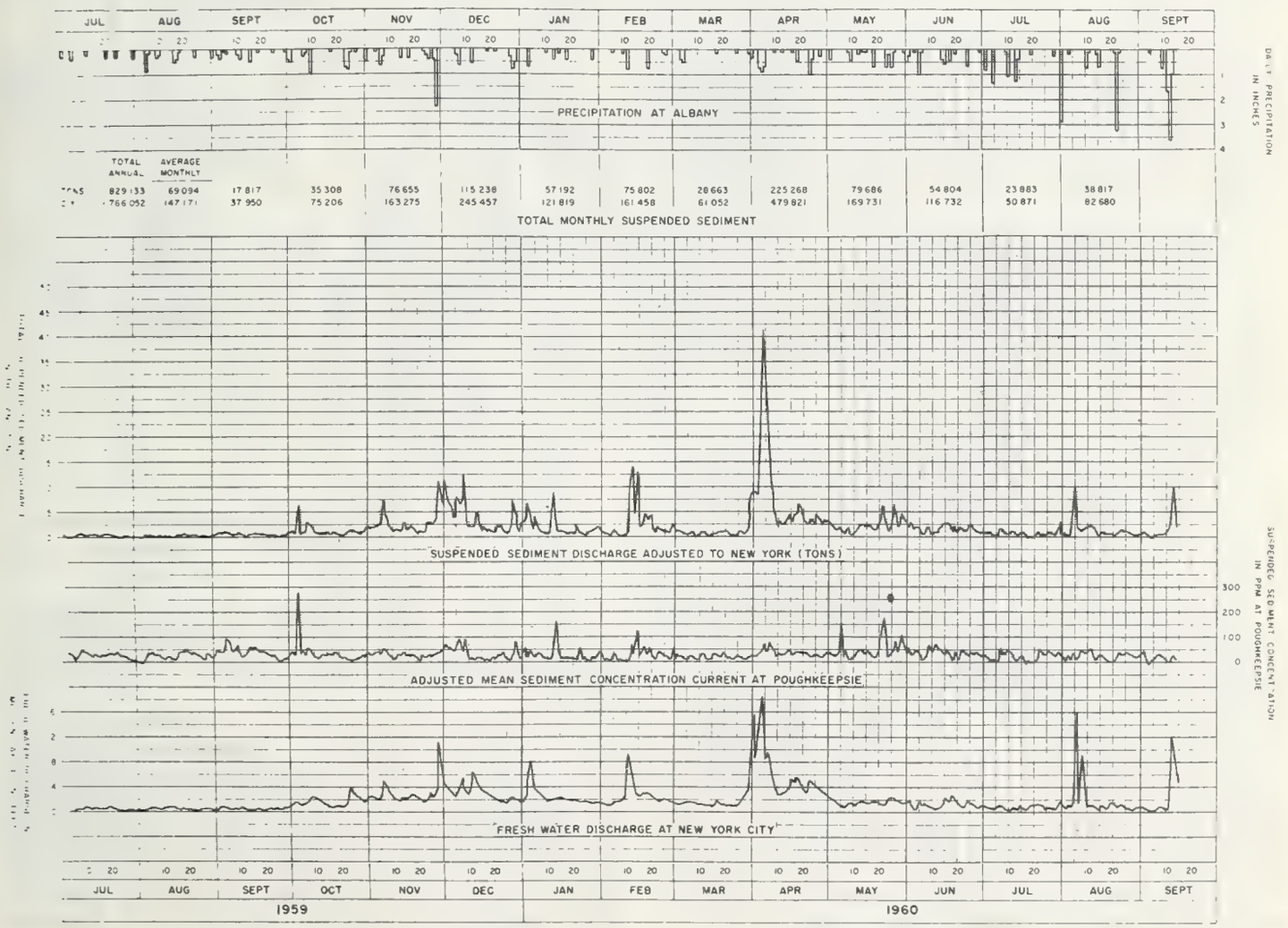


FIGURE 9. — Lower Hudson River sediment measurements, Poughkeepsie, N. Y.

Based on the foregoing, the total annual suspended sediment load transported to the Lower Hudson River for the 1-year period, September 1959 through August 1960, amounts to about 829,000 tons, or 1,766,000 cubic yards, based on an average dry density of 939 pounds per cubic yard. The total monthly loads are shown on figure 9. The total measured suspended sediment load is representative only of the period studied and cannot be considered as the average yearly load for the Hudson River. Even though the yearly fresh water discharge compares closely with the long term discharge, the sediment concentration varies considerably.

An analysis of the sedimentation measurements and computations result in the following observations:

1. The fresh water discharge at Green Island ranged from a low of 1,340 c.f.s. (5 July 1959) to a high of 216,896 c.f.s. (15 April 1960) with a period average of 16,505, which compares favorably with the 12-year mean of 14,400 c.f.s.

2. The mean daily suspended sediment concentration ranged from a low of 3 p.p.m. on 9 February 1960 to a high of 267 p.p.m. on 4 October 1959, for a daily average of 33 p.p.m.

3. The daily suspended sediment varied from a low of 59 tons (126 cu. yd.) on 28 September 1959 to a high of 41,300 tons (88,000 cu. yd.) on 5 April 1960 for an average of 2,272 tons (4,840 cu. yd.). Lower suspended sediment flows occur during the summer and fall months and higher sediment flows during the winter and spring.

4. High river discharges generally contain high sediment concentration.

5. The highest recorded sediment concentration was observed during a low flow, most probably due to an intense rainfall pattern of short duration over a particularly erodible area of the watershed.

Undisturbed samples of shoal material were obtained from the six major shoal areas and six pier slips where the silting is extremely heavy. These samples were analyzed to determine the in-place wet and dry densities and to identify the major chemical components and grain size of the shoal material for comparison with the upstream sediment samples obtained at Poughkeepsie. Fifty samples were obtained in the locations indicated on figure 10. At least two samples were taken in each shoal area. The samples were taken in the vertical with a 3-inch Charleston-type window sampler at depths ranging from 2 to 12 feet below the bottom, the depth to which the sampler penetrated due to its own weight. The Savannah fish-type sampler was used to obtain the "soupy" material at the top of the shoal, which was too fine or fluid to be collected by the window-type sampler.

The shoal samples were analyzed to determine the wet and dry densities, water content, specific gravity, and grain-size distribution (table 1). Twelve suspended sediment samples, taken at Poughkeepsie, N. Y. were similarly analyzed (table 1). The dry and wet densities were found to be generally constant in the vertical and not to increase with the depth. The average of the in-place dry densities of the upper layers of bottom material varied from 38.46 pounds per cubic foot in the channel to 31.12 pounds per cubic foot

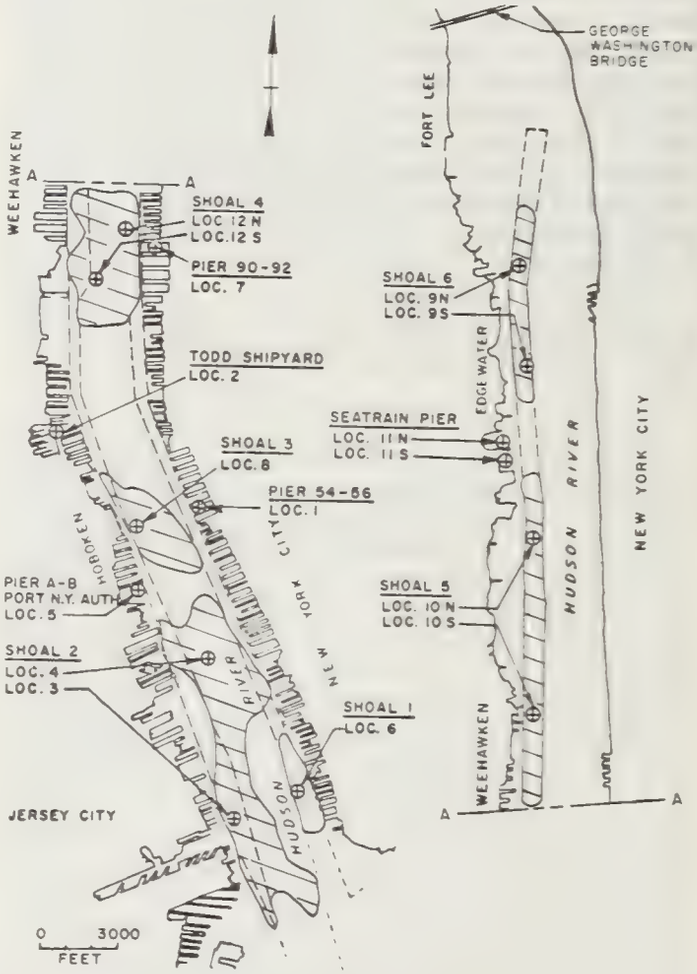


FIGURE 10. — Location of shoaling sampling in lower Hudson River.

in the slips. The average of these densities, 34.79 pounds per cubic foot, was used to convert total suspended sediment weight to volumetric units.

In order to identify further the similarity of shoal material and the upstream sediment source, grain size distribution was determined by mechanical analysis of the following: bottom samples taken from the channel shoal areas in 1960, a 180-gallon composite water-sediment sample taken at Poughkeepsie during 1959-60, and hopper dredge bin samples of material dredged from the Hudson River 30-foot project

TABLE 1.—*Physical and mechanical properties of Hudson River channel and slip samples and Poughkeepsie samples*

Location of sample	Depth of shoal surface	Depth of sample below shoal surface	Density			Wt. of dry solids	Analysis					
			Wet		Dry (salt-free)		Water content	Specific gravity	Organics	Gravel	Sand	Fines (clay or silts)
	Ft. m.l.w.	*Ft.*	*G/cc.*	*(Lb./ cu. ft.)*	*(Lb./ cu. ft.)*	*Pct.*	*Pct.*		*Pct.*	*Pct.*	*Pct.*	*Pct.*
Shoal 1, location 6	45.0	*2	1.430	89.23	44.74	50.14	85.1	2.62	9.2	15	34	51
Shoal 2, location 3	37.1	4	.....	.....	.....	.....	174.3	2.53	12.0	0	1	99
		6	1.302	81.24	45.18	55.61						
		8	1.494	93.23	53.13	56.99						
		10	1.507	94.04	24.94	26.52						
Shoal 2, location 4	48.0	*2	1.515	94.54	48.19	50.97	75.9	2.70	6.3	0	25	75
Pier A-B, N.J., location 5, Port of N.Y. Authority	26.1	**+0.1	1.244	77.62	26.08	33.60						
		1.9	1.318	82.24	30.80	37.45						
		3.9	1.245	77.69	29.60	38.10						
		5.9	1.186	74.01	25.52	34.48						
		7.9	1.225	76.44	27.83	36.41						
Piers 54-56, N.Y., location 1	24.7	**+0.4	1.201	74.94	22.11	29.50						
		1.6	1.326	82.74	32.86	39.71						
		3.6	1.228	76.62	28.00	36.54						
		5.6	1.231	76.81	28.30	36.84						
		7.6	1.275	79.56	35.50	44.62						
		9.6	1.265	78.90	19.67	24.93						
Shoal 3, location 8	50.2	3	.....	.....	.....	.....	108.8	2.70	7.3	18	13	69
		5	1.394	86.98	39.51	45.42						
Todd Shipyards, location 2	25.8	**2.5	1.264	78.87	29.90	37.91						
		**4.5	1.241	77.44	28.60	36.93						
		6.5	1.274	79.50	33.00	41.51						
		8.5	1.303	81.31	30.30	37.26						
		10.5	1.365	85.18	37.78	44.35						
Shoal 4, location 12N	51.1	*2.2	1.458	90.98	41.95	46.11	75.1	2.72	5.5	0	2	98
Shoal 4, location 12S	52.4	*0	1.381	86.17	37.30	43.29						
		2	1.489	92.91	48.71	52.43						
Piers 90-92, N.Y., location 7	43.8	**+0.8	1.178	73.50	50.70	68.98						
		3.2	1.251	78.06	26.72	34.23						
		5.2	1.309	81.68	32.86	40.23						
		7.2	1.270	79.25	34.80	43.91						
		9.2	1.298	80.99	33.47	41.33						
Shoal 5, location 10S	40.0	*3	1.336	83.37	25.84	30.99	77.4	2.71	7.3	0	5	95
Shoal 5, location 10N	33.5	*0	1.441	89.92	58.51	65.07						
		*2	1.459	91.04	34.37	37.75						
Seatrain Pier, location 11S	20.7	**1.5	1.263	78.81	25.46	32.30						
		3.6	1.300	81.12	31.01	38.23						
		5.6	1.289	80.43	28.38	35.28						
		**2	1.338	83.49	34.66	41.54						
		**4	1.317	82.18	31.54	38.37						
Seatrain Pier, location 11N	27.4	6	1.294	80.74	30.36	37.60						
		8	1.335	83.30	36.40	43.70						
		10	1.323	82.55	33.36	40.41						
		12	1.319	82.30	31.73	38.55						
		*2	1.383	86.30	35.50	41.13	132.2	2.70	10.9	0	2	98
		*4.5	1.291	80.56	30.17	37.45						
Shoal 6, location 9S	32.5	6.5	1.360	84.86	37.07	43.68						
		8	1,392	86.86	39.64	45.64						
		10	1.372	85.61	35.38	41.33						
		*+0.2	1.274	79.50	30.39	38.23						
Shoal 6, location 9N	.....	1.8	1.414	88.23	41.73	47.30						
		2.8	1.368	85.36	39.35	46.10						
Average	.....	.....	.....	.....	.....	.....	104.1	2.67	8.4	5	12	83
Average, channel*			1.404	87.58	38.46	43.91	.....	.....	.....	.....	.....	.....
Average, slips**			1.244	77.64	31.12	40.08	.....	.....	.....	.....	.....	.....
Average, slips and channels			1.324	82.61	34.79	42.11	.....	.....	.....	.....	.....	.....
Average, Poughkeepsie waterworks (12 samples)			1.004	62.63	.111	.18	142.0	2.53	2.28	0	8	92
Average, 30-ft. project (1952-57)			1.427	89.04	.....	.....	111.1	2.63	9.8	0	6	94
Average, 40-ft. project (1954-55)			1.463	91.29	.....	.....	108.0	2.64	.....	0	15	85

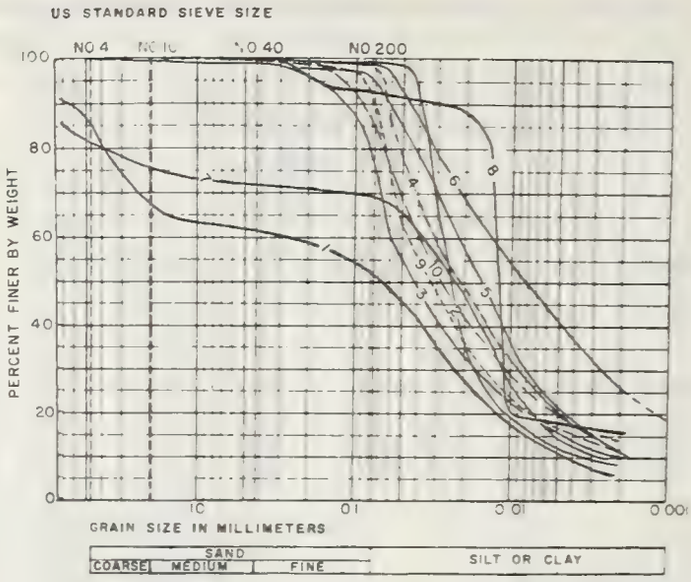
channel in Weehawken-Edgewater between 1952 and 1957 and the Hudson River 40-foot project channel in 1954-55. The results, summarized in table 2 and shown in figure 11, indicate that the Poughkeepsie sediments are generally finer than the channel shoal materials. In the upper particle-size range the shoal samples show a large amount of coarser materials. However, in the lower particle-size range, the shoal material is similar to the Poughkeepsie sediments. Overall, the lower Hudson River shoal materials have physical characteristics similar to those of the Poughkeepsie sediments, indicating a definite upstream source of sediment.

Standard chemical tests were made on bottom samples taken from the slip and channel shoals and the composite 180-gallon water-sediment sample at Poughkeepsie. The results are shown in table 3. In addition, a spectographic analysis was made of the Poughkeepsie composite sample and a sample taken from the south side of the Seatrain Pier at Weehawken with the following results.

Seatrain slip, Weehawken, southside: over 10 percent silicon; 1 to 10 percent calcium, aluminum, magnesium, and iron; 0.1 to 1 percent sodium. Not detected, potassium.

Poughkeepsie composite: over 10 percent iron, silicon; 1 to 10 percent calcium, aluminum, and magnesium; 0.01 to 1 percent sodium. Not detected, potassium.

Comparison of these test results, while not



LOCATION: HUDSON RIVER SHOALS & POUGHKEEPSIE

SAMPLE NO	LOCATION	DATE
1	SHOAL NO 1, LOC 6	JULY 13, 1960
2	SHOAL NO 2 LOC 3	JULY 7 1960
3	SHOAL NO 2 LOC 4	JULY 7, 1960
4	SHOAL NO 4 LOC. 12N	JULY 13, 1960
5	SHOAL NO 5 LOC. 10S	JULY 12, 1960
6	SHOAL NO 6 LOC 9S	JULY 11, 1960
7	SHOAL NO 3 LOC 8	JULY 7, 1960
8	POUGHKEEPSIE WATER WORKS	OCT. 59-JULY 60
9	WEEHAWKEN-EDGEWATER 30 FT CHANNEL	1952-1957
10	HUDSON RIVER 40 FT CHANNEL	1954-1955

FIGURE 11. — Grain-size distribution curve.

TABLE 2.—*Comparative grain size of samples from Poughkeepsie, Hudson River shoals 1 to 6, and 30-foot and 40-foot project channels*

Place and location of sample	Date	Grain-size (millimeters)				Diameter size	
		0.001 to 0.01	0.01 to 0.1	0.1 to 1.0	1.0 and Over	D_{50}	D_{90}/D_{10}
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Mm.*	*Mm.*
Poughkeepsie waterworks intake: Composite samples (fines 92 percent, sand 8 percent)	Oct. 59–July 60	20	73	7	0	0.013	1.5
Project channels, shoal areas:							
Shoal No. 1 Location 6	July 60	18	37	8	37	.066	308.0
2 3	July 60	19	80	1	0	.025	3.1
2 4	July 60	23	64	12	1	.041	10.0
3 8	July 60	27	43	3	27	.023	571.0
4 12	July 60	32	66	2	0	.021	10.2
5 10	July 60	35	62	3	0	.016	9.1
6 9	July 60	55	43	2	0	.008	3.0
Average (fines 83 percent, sand 12 percent, gravel 5 percent)	.....	30	57	4	9	.027	7.0
Edgewater-Weehawken-Hudson River, 30-foot project channel:							
Hopper dredge bin	1952	30	64	6	0	.019	8.0
Hopper dredge bin	1953	29	69	2	0	.026	11.2
Hopper dredge bin	1956	19	77	4	0	.040	5.0
Hopper dredge bin	1957	36	62	2	0	.030	20.0
Average, 12 samples (fines 94 percent, sand 6 percent)	.....	28	68	4	0	.033	8.7
Hudson River, 40-foot project channel: Average, 14 samples (fines 85 percent, sand 15 percent)	1954–55	29	65	6	0	.029	9.4

TABLE 3.—*Chemical analysis of upstream and downstream shoaling material, 1960, and composite at Poughkeepsie, 1959–60*

Sample Location	Sample Date	Chlorides	Sulfates	Alkalinity methyl orange	Carbonate hardness	Non-carbonate hardness	Iron soluble	Silicon dioxide	Aluminum
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Shoal 1	July 13	0.635	0.337	0.152	0.152	15.078	0.00300	52.000	1.055
Shoal 2, south	July 7	.834	.257	.097	.097	16.454	.00450	23.600	1.682
Shoal 2, north	July 7	.612	.308	.180	.180	16.182	.01200	46.000	1.468
Between piers A-B, N.J.	July 7	.952	.007	.107	.107	4.126	.00019	8.260	.230
Between piers A-B, N.J.	July 7	.750	.148	.097	.097	15.297	.01250	23.200	1.582
Shoal 3	July 7	.690	.216	.097	.097	15.966	.00300	43.400	1.638
Between piers 54-56	June 7	1.166	.075	.074	.074	2.639	.00007	4.700	.185
Between piers 54-56	June 7	.875	.453	.111	.111	14.828	.00400	30.000	1.557
Between piers 16-H, N.J.	June 7	.815	.108	.025	.025	2.298	.00015	.812	.080
Between piers 90-92, N.Y.	July 7	1.202	.043	.118	.118	2.596	.00008	4.890	.160
Between piers 90-92, N.Y.	July 7	.850	.123	.180	.180	11.996	.00750	20.900	1.211
Shoal 4, north	July 13	.472	.230	.097	.097	18.038	.01150	48.300	1.772
Shoal 5, south	July 12	.547	.288	.070	.070	17.694	.01350	44.200	1.720
Seatrain pier, south	July 8	.544	.158	.097	.097	16.741	.00350	23.300	1.700
Seatrain pier, north	July 8	.529	.009	.066	.066	11.801	.00044	20.250	1.177
Seatrain pier, north	July 8	.547	.070	.111	.111	15.104	.00750	30.700	1.542
Shoal 6, south	July 11	.373	.123	.111	.111	15.300	.00500	44.200	1.490
Shoal 6, south	July 11	.662	.015	.077	.077	11.558	.00021	15.700	1.254
Average	.....	.725	.165	.104	.104	12.428	.005	26.912	1.195
Composite, Poughkeepsie	Oct. 59 to July 60	.025	.029	.097	.097	32.629	.020	16.600	.952

Sample Location	Sample Date	Iron total	Calcium	Magnesium	Sodium	pH[1]	Depth of sample below top of shoal
		Pct.	*Pct.*	*Pct.*	*Pct.*		
Shoal 1	July 13	1.421	1.740	0.290	0.161	6.7	2 ft. below bottom.
Shoal 2, south	July 7	1.747	.303	.422	.177	6.5	4 ft. below bottom.
Shoal 2, north	July 7	1.557	.944	.403	.155	6.7	2 ft. below bottom.
Between piers A-B, N.J.	July 7	.695	.109	.222	.172	7.4	.5 ft. above bottom.[2]
Between piers A-B, N.J.	July 7	1.601	.322	.385	.150	6.9	.1 ft. above bottom.
Shoal 3	July 7	1.748	.172	.447	.149	6.8	3 ft. below bottom.
Between piers 54-56	June 7	.327	.064	.156	.459	7.5	.5 ft. below bottom.[2]
Between piers 54-56	June 7	1.523	.332	.329	.209	6.7	1.6 ft. below bottom.
Between piers 16-H, N.J.	June 7	.056	.024	.074	.427	7.0	.5 ft. below bottom.[2]
Between piers 90-92, N.Y.	July 7	.370	.062	.164	.430	7.6	.5 ft. above bottom.[2]
Between piers 90-92, N.Y.	July 7	1.288	.237	.319	.160	6.8	.8 ft. above bottom.
Shoal 4, north	July 13	1.995	.286	.536	.104	6.5	2.2 ft. below bottom.
Shoal 5, south	July 12	1.937	.349	.517	.100	6.3	3.0 ft. below bottom.
Seatrain pier, south	July 8	1.893	.209	.455	.120	6.5	3.6 ft. below bottom.
Seatrain pier, north	July 8	1.362	.151	.311	.378	7.3	.5 ft. below bottom.[2]
Seatrain pier, north	July 8	1.768	.172	.355	.110	6.7	2 ft. below bottom.
Shoal 6, south	July 11	1.870	.226	.382	.088	6.8	2 ft. below bottom.
Shoal 6, south	July 11	1.160	.111	.309	.461	7.4	10 ft. below bottom.
Average	.....	1.351	.323	.338	.501	6.9	
Composite, Poughkeepsie[3]	Oct. 59 to July 60	9.860	.097	.169	.020	6.6	

[1] 20 percent solution.
[2] Sample obtained with fish type sampler.
[3] Potassium not detected.

entirely conclusive, indicated a reasonable similarity between the sediments from the upstream Hudson River drainage area and the shoaling material found in the lower Hudson River.

Sanitary and storm sewer and industrial and other waste material are discharged or dumped into the Hudson River and other waterways tributary to the Harbor.

An inventory of data from local agencies revealed that 159,000 cubic yards of dry solid industrial waste and sewage have been discharged annually into the Hudson River between the Battery, Manhattan, and Bear Mountain, and 473,000 cubic yards of dry solids into other portions of New York Harbor for a total source of 632,000 cubic yards annually. Although a part of these solids would be reduced by natural processes and another part would never settle to the bottom due to its specific gravity, the rest constitutes a consideral potential source of shoal material in the harbor. This source of shoal material is estimated at 412,000

cubic yards, about 65 percent of the annual discharge or about 10 percent of the annual shoaling in the Lower Hudson River, including the slips. Although these studies do not represent a complete inventory and accounting of material source into the Lower Hudson River, certain general findings are evident.

The Hudson River watershed is a definite source of sediment moving into the Lower Hudson River. This source represents an estimated potential shoaling material equal to about 1¾ million cubic yards annually. This represents a suspended sediment load of 43 percent of the annual shoaling in the Lower Hudson River. Some of these sediments would probably settle in the Haverstraw Bay and Tappan Zee area because of predominant upstream bottom currents north of the George Washington Bridge during medium and low fresh waterflows in the Hudson River. However, it is reasonable to consider that losses in this area would be offset by sediment discharge entering the Lower Hudson River.

These two sources do not account for all the sediment deposited in shoals. Measurements were not made of other possible sources, such as the 3,000-square mile area tributary to the New York Harbor downstream of the Hudson River and the Atlantic Ocean on the floodtides.

Cross-Sectional Distribution of Velocity, Salinity, and Sediment Discharge

For a better evaluation of the factors that influence the shoaling and to obtain prototype data for construction and verification of the models, tidal observations, velocity measurements and water samples for salinity, and sediment concentration, distribution, and discharge were taken at stations established on each of five ranges (fig. 12). These ranges were located across the river, spaced about 2 miles apart along the 11-mile reach of the Lower Hudson River between the Battery and George Washington Bridge. A single master control station was established on range A, the farthest downstream, and three stations were established at each of the other ranges, B through E.

Ideally, the measurements were designed for a single complete tidal cycle during average conditions for all stations, simultaneously. However, since this was not possible because of field conditions, personnel, and equipment, the measurements were made on successive days beginning with range B on 23 April 1957 and continuing for one tidal cycle on each of the upstream ranges in succession until range E was completed on 26 April. The master control station was operated throughout the time that the measurements were performed on any of the other stations, to measure the similarity of

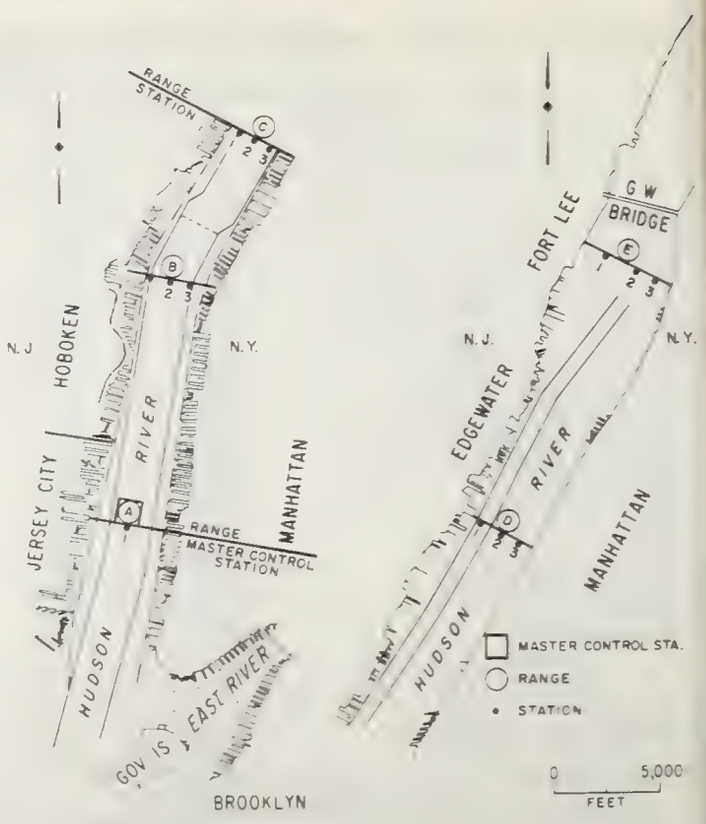


FIGURE 12. — Location of ranges in lower Hudson River.

conditions under which the measurements for each station were taken.

The measurements were taken from small pontoons anchored at each station. The field party on each of the four pontoons, three on the upstream range and one on the control station, consisted of two men. The work was performed only on alternate tidal cycles, and the men worked in two shifts of ten men each to take advantage as much as possible of daylight hours. Velocity measurements were obtained by Price current meter. Steel vanes suspended from a cable served as current direction indicators. Pint-size milk bottles and a mechanism that permitted opening of the bottles at the desired depth were used to obtain water samples.

At each range, the velocity and direction of current were measured simultaneously at the three stations and the master control station at half-hour intervals for 5 depths: 1.5 feet below the surface, at ¼, ½, and ¾ depths, and 2 feet above the bottom. Water samples were collected hourly at each station for every depth simultaneously with the velocity measurements. The results of these measurements in terms of cross-sectional distribution of velocity, salinity, tide level, sediment discharge at minimum, mid, and maximum velocity are shown for each range (B through E) on figures 13, 14, and 15. The vertical variation in velocity, salinity, and sediment in the midstation of range A over a

tidal cycle is shown on figure 16. The vertical and horizontal variation of sediment discharge across range B during one tidal cycle is shown on figure 17. The variation in the average velocity, salinity, and sediment at midstation of range A and at range E over a tidal cycle is shown on figures 18 and 19. The net sediment discharge in cubic feet per second for all ranges (A to E) is shown in table 4.

The cross-sectional channel geometry varies

TABLE 4.—*Lower Hudson River: net sediment discharge, April 23 to 26, 1957* [1]

Range	Station	Date	Sediment discharge at indicated depth					Net sediment total
			Top −1.5 ft.	¼ depth	½ depth	¾ depth	Bottom +2 ft.	
			C.f.s.	*C.f.s.*	*C.f.s.*	*C.f.s.*	*C.f.s.*	*C.f.s.*
A	2	23	−16.99	− 4.60	+ 5.24	+17.50	+ 3.30	+ 4.45
	2	24	−30.38	−30.94	− 3.48	+ 1.10	+ 6.99	−56.71
	2	25	−39.07	−65.20	+17.33	+20.76	+ 6.85	−59.33
	2	26	−14.94	−18.25	+16.33	+ 8.72	+ 2.03	− 6.11
B	1	23	− 3.94	− 3.91	+ 2.97	− 0.47	− 0.15	− 5.50
	2	23	− 8.29	− 7.46	− 2.05	+ 0.88	− 0.41	−17.33
	3	23	− 0.42	− 6.87	− 3.63	+ 0.71	+ 2.92	− 7.29
Total			−12.65	−18.24	− 2.71	+ 1.12	+ 2.36	−30.12
C	1	24	+ 2.60	+ 3.72	+ 0.54	+ 2.66	+ 2.38	+11.90
	2	24	− 1.75	− 1.93	− 2.91	+ 1.51	+ 0.05	− 5.03
	3	24	− 8.66	− 1.15	− 3.00	− 0.44	− 0.52	−13.77
Total			− 7.81	+ 0.64	− 5.47	+ 3.83	+ 1.91	− 6.90
D	1	25	− 9.24	− 2.96	− 0.97	+ 0.89	+ 1.46	−10.91
	2	25	− 1.66	+ 1.42	+ 4.46	+ 6.25	+ 0.67	+11.14
	3	25	− 1.12	− 3.10	+ 5.72	+ 2.55	+ 0.55	+ 4.60
Total			−12.02	− 4.64	+ 9.21	+ 9.60	+ 2.68	+ 4.83
E	1	26	− 1.64	− 5.57	− 7.46	− 0.86	− 1.82	−17.35
	2	26	+10.00	− 9.85	− 1.82	+ 1.61	+ 1.35	+ 1.29
	3	26	− 4.28	+ 2.94	+ 7.20	+ 4.50	+ 2.34	+12.70
Total			+ 4.08	−12.48	− 2.08	+ 5.25	+ 1.87	− 3.36
Total of stations B–E			−28.40	−34.72	− 1.05	+19.80	+ 8.82	−35.55
Average			− 7.10	− 8.68	− 0.26	+ 4.95	+ 2.21	− 8.88

[1] − = downstream flow; + = upstream flow.

Average discharge for 11 years, Green Island, N.Y. = 13,930 c.f.s.

Average discharge for April 1957 = 15,730 c.f.s.

Computation of annual sediment yield = $\frac{-8.88 \text{ c.f.s.} \times 43{,}300 \text{ sec. per cycle}}{27 \text{ (cu. ft. per cu. yd.)}} = 14{,}200$ cu. yd. per cycle

14,200 × 700 cycles = 9,940,000 cu. yd./yr.

from a horizontal bottom in ranges B and C to the gorgelike bottom on the New York side in ranges D and E (fig. 13). The part of the river downstream of range C is maintained to depths of 40 to 48 feet by dredging practically throughout its width to the pierhead lines, whereas the river upstream has a natural deep section along the New York side and a shallow depth on the New Jersey side in which a 30-foot navigable channel is maintained.

The velocities generally decrease with the depth (fig. 16) and are higher on the New York side except for the upstream range E (figs. 13 to 15). However, the maximum velocities are about equal. The velocities are higher during the ebbtide than during the floodtide (figs. 18-19) with the maximum ebb velocity recorded at 5.7 feet per second and with the maximum flood velocity recorded at 4.0 feet per second. The slack current lags the slack water by about 3 hours at ebb and floodtides (figs. 18 and 19).

The salinity increases with depth and varies at some time as much as 22,000 p.p.m. from top to bottom (figs. 13 to 16). The saline density layers for all ranges of velocities generally increase toward the bottom and the New York side of the river, not only in the upstream ranges where the natural deep water is on the New York side but also in the downstream ranges where the depth is uniform across the river (figs. 13 to 15).

The greatest sediment discharge for all periods of flow was found in suspension in the upper half of the depth where not only is the sediment concentration fairly high but also the velocities are closest to maximum (figs. 13

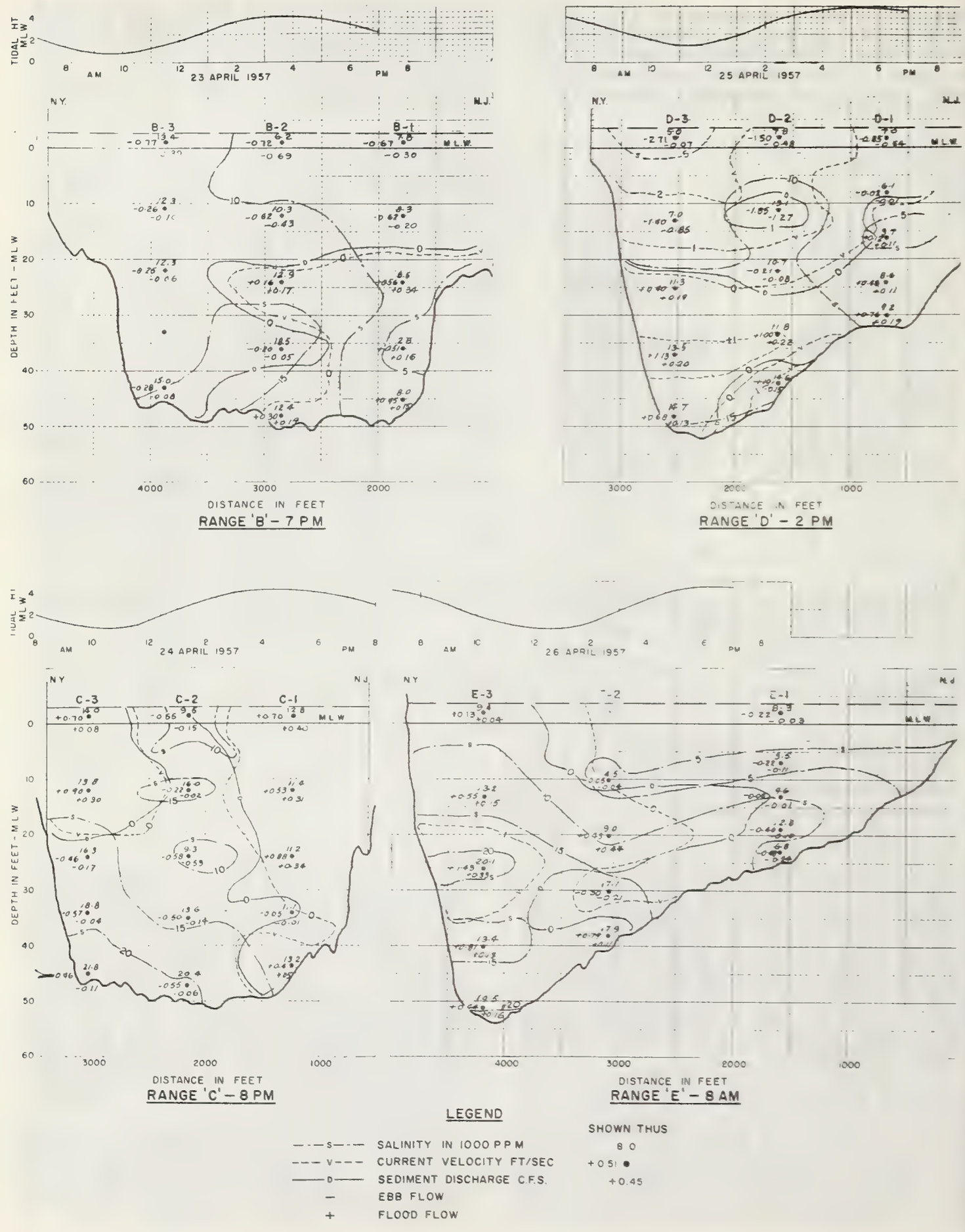


FIGURE 13. — Current velocity, sediment discharge, and salinity at minimum velocity.

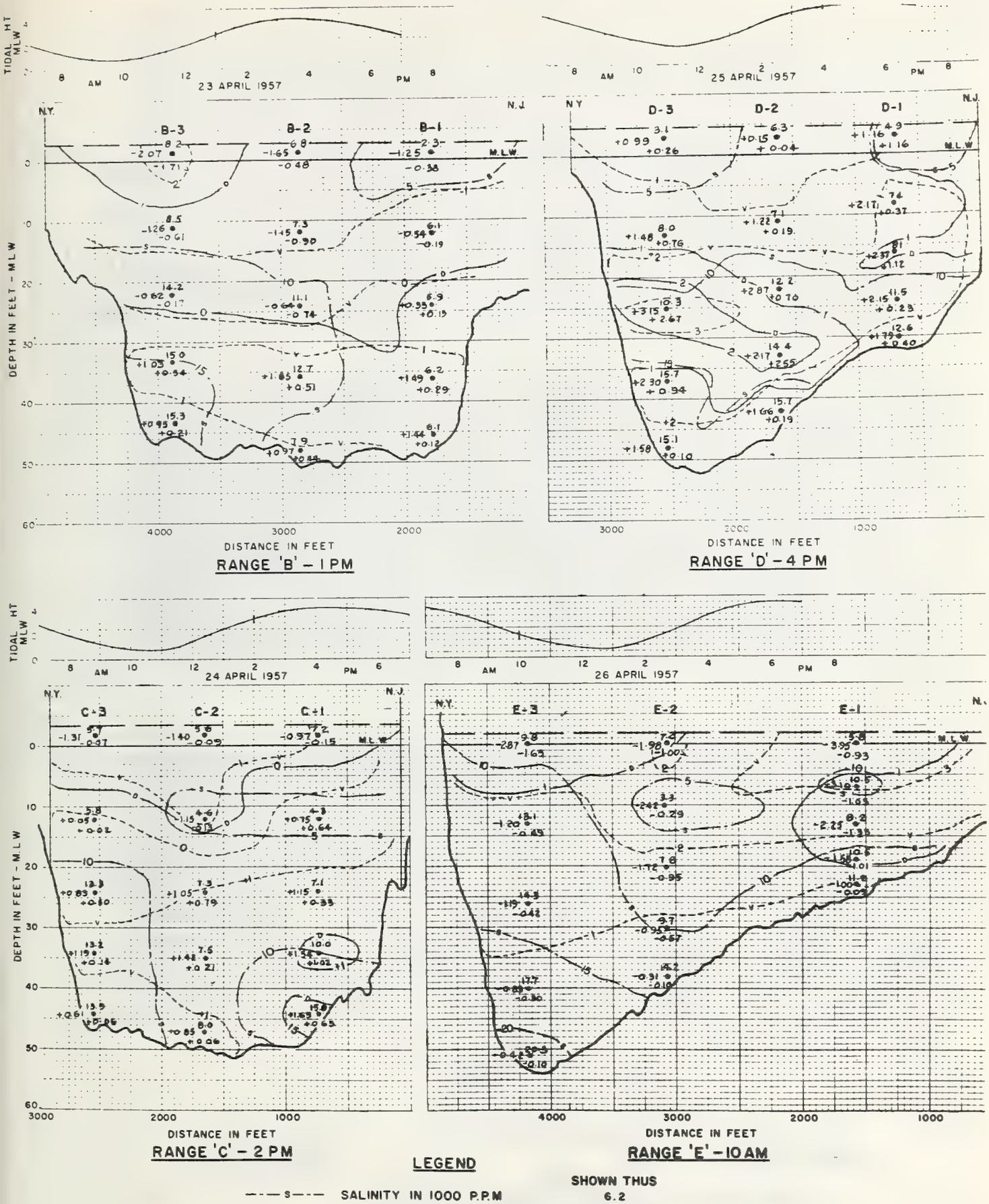


FIGURE 14. — Current velocity, sediment discharge, and salinity at mid-velocity.

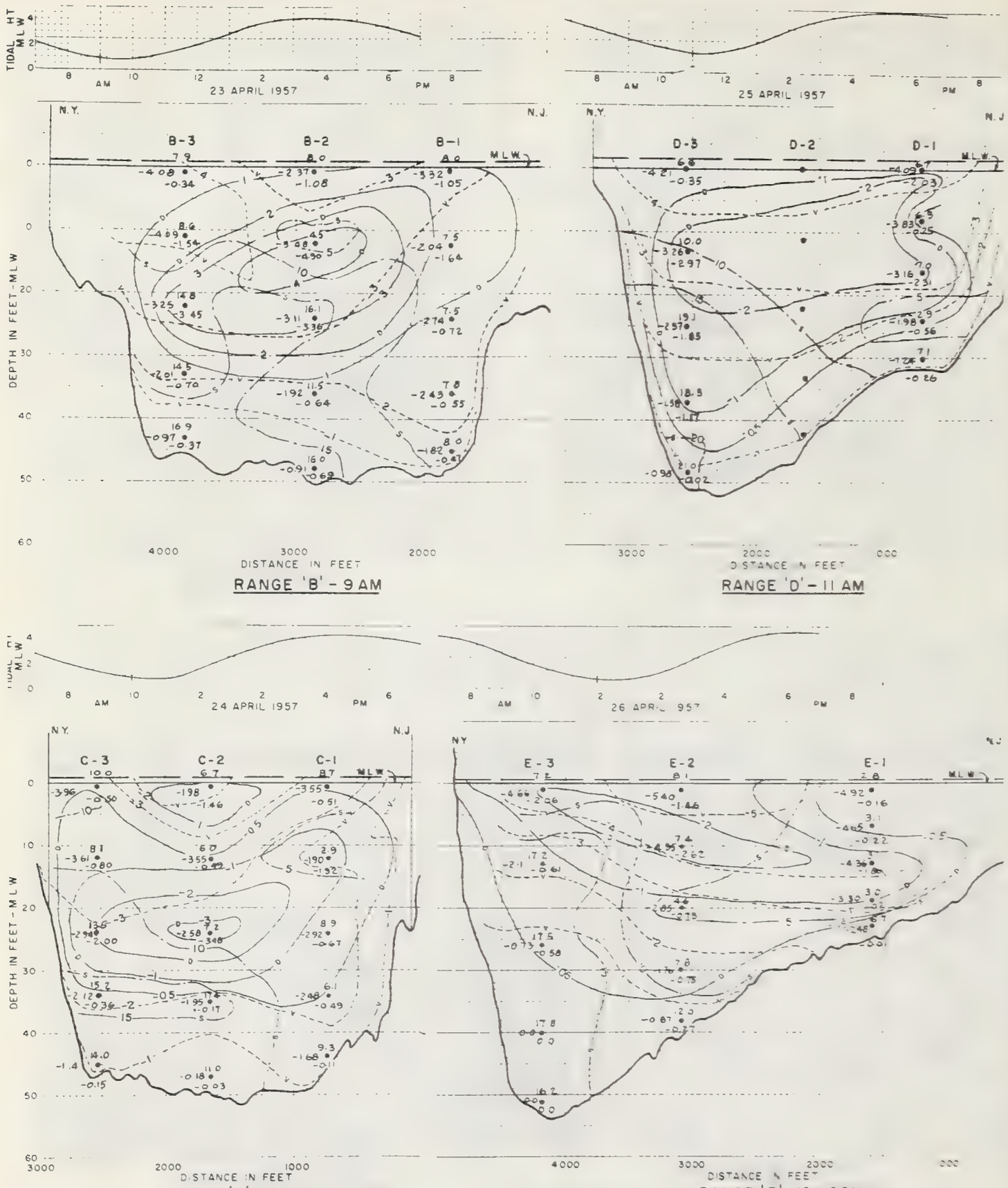


FIGURE 15. — Current velocity, sediment discharge, and salinity at maximum velocity.

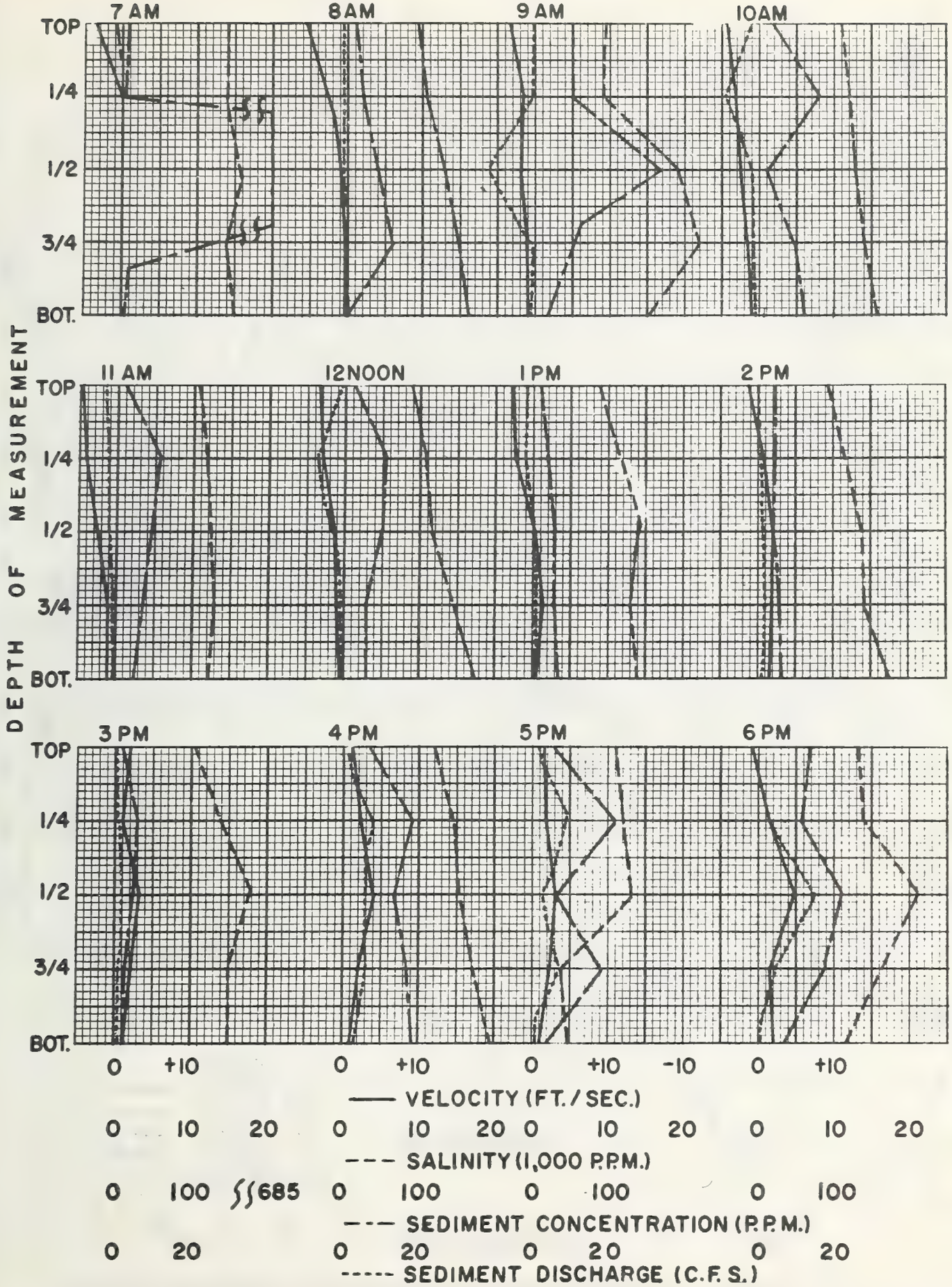


FIGURE 16. — Vertical variation in velocity, salinity, and sediment, lower Hudson River range A, 23 April 1957.

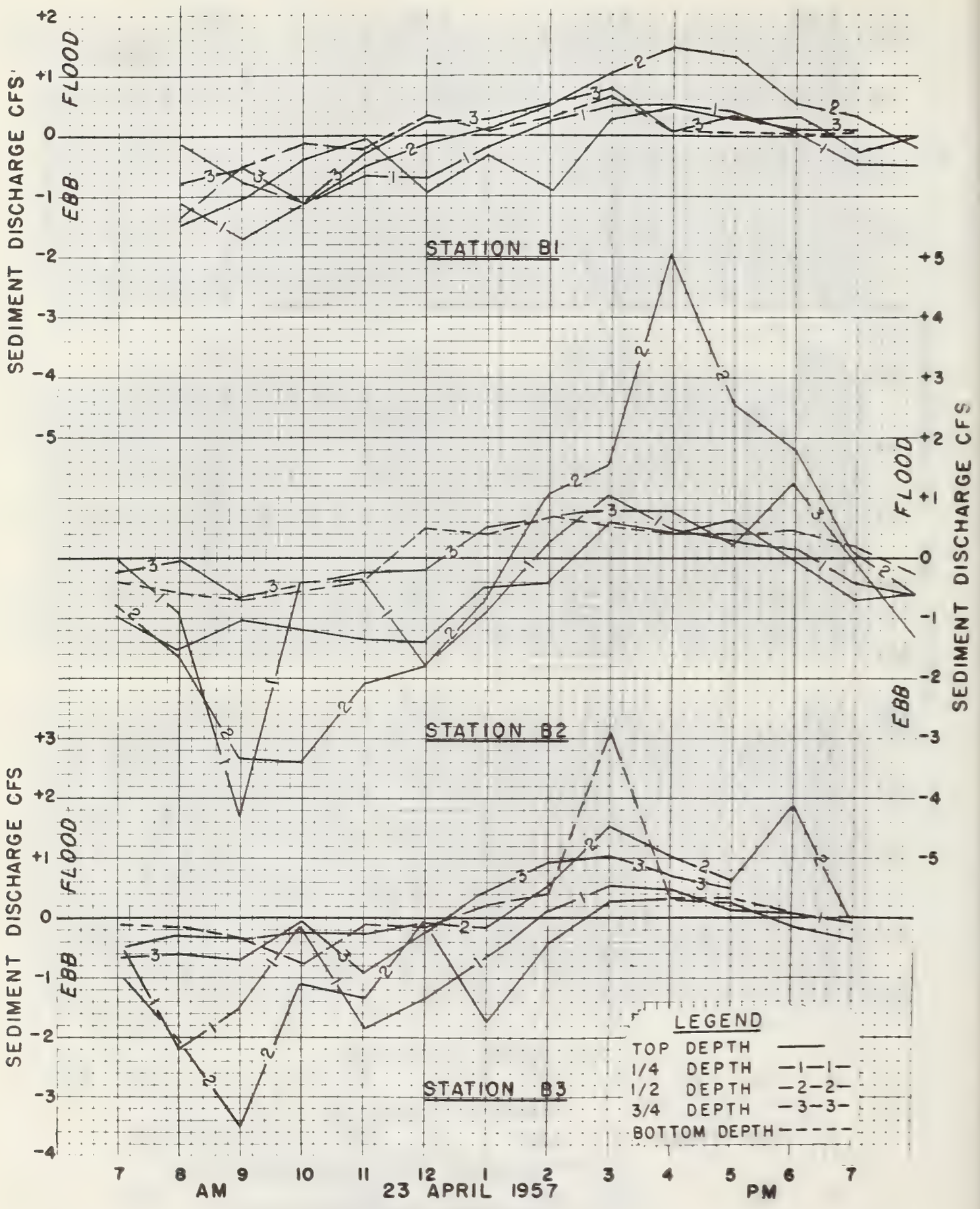


FIGURE 17.—Vertical and horizontal variation in sediment discharge over one tidal cycle.

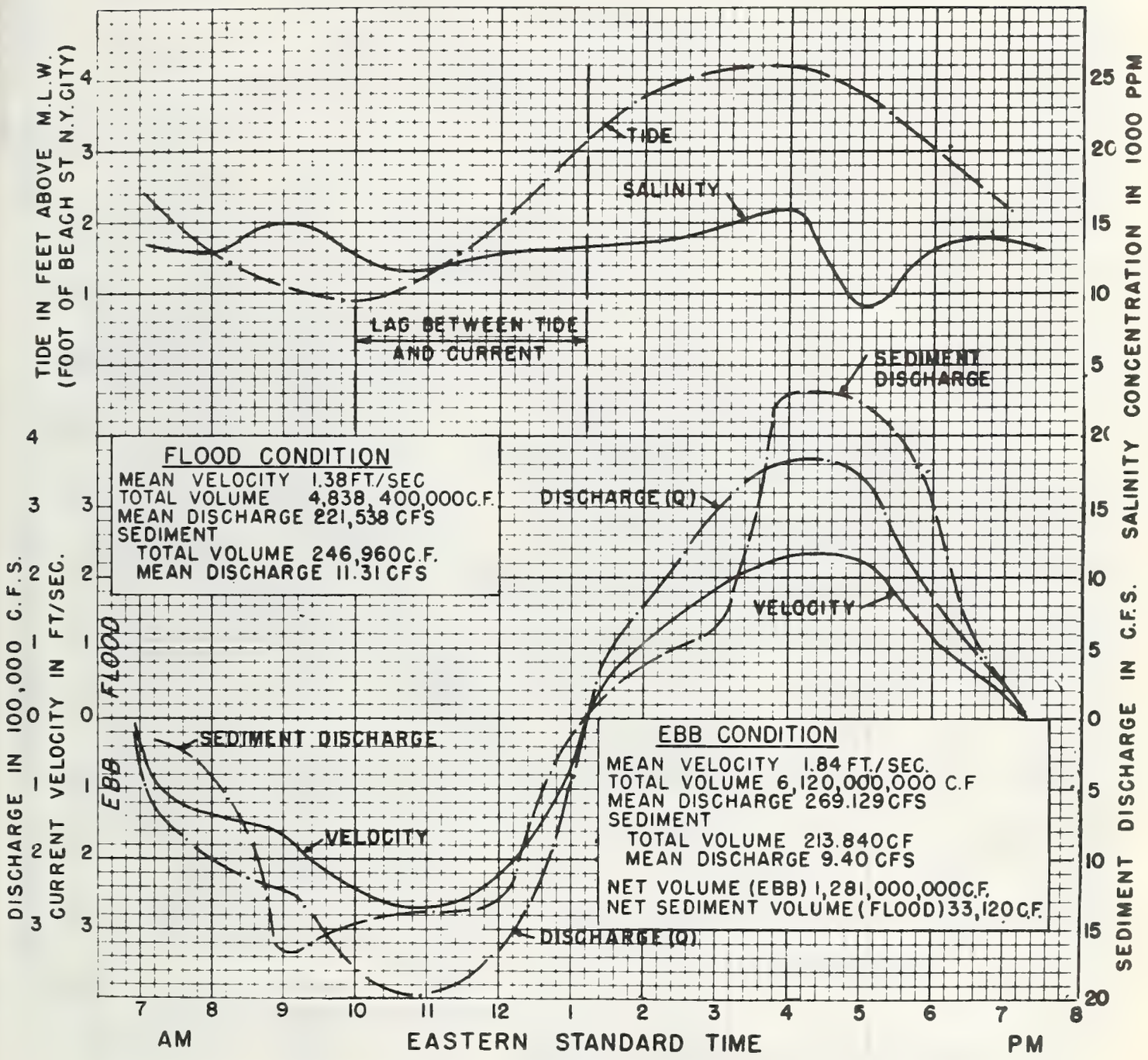


FIGURE 18.—Variation in average velocity, salinity, and sediment, range A, 23 April 1957.

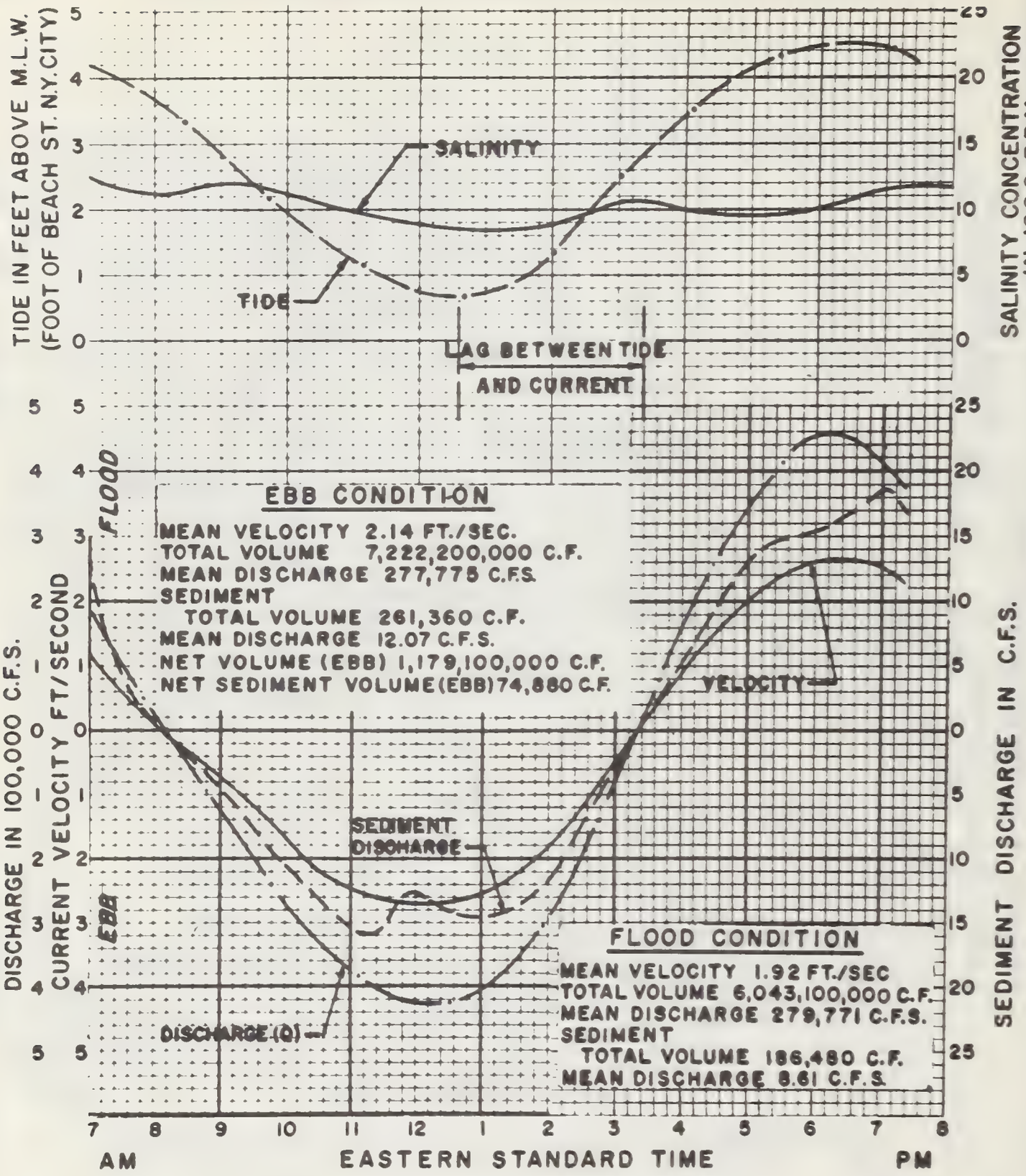


FIGURE 19. — Variation in average velocity, salinity, and sediment, range E, 26 April 1957.

through 17). During the time when the currents are changing from ebb to flood, with the ebb current still flowing on the top and the flood currents flowing in on the bottom, a definitely greater concentration of sediment is found in the bottom layers. It was also found that the greater sediment concentration existed in the boundary zone between the salt and brackish layer (table 5).

The sediment discharge was determined by

TABLE 5.—*Typical prototype observations and computations of salinity, velocity, and sediments, Lower Hudson River at range B, station 1, April 23, 1957*[1]

Current meter depth	Weighted area	Velocity	Q	Sediment concentration	Sediment discharge		Salinity
Depth 1.5 feet from top:	*Ft.*[2]	*Ft./sec.*	*C.f.s.*	*P.p.m.*	*Lb./sec.*	*C.f.s.*	*P.p.m.*
7 a.m.	×	—	—	—	—	—	—
8	7,320	−3.41	−24,961	54.9	− 85.5	−1.37	7,600
9	6,150	−3.32	−20,418	51.4	− 65.5	−1.05	8,000
10	6,150	−3.63	−22,324	17.2	− 24.0	− .38	6,700
11	7,080	−3.35	−23,718	2.6	− 3.8	− .06	4,000
12	8,720	−2.40	−20,928	45.1	− 58.9	− .94	1,200
1	10,830	−1.25	−13,538	27.9	− 23.6	− .38	2,300
2	12,700	− .84	−10,668	82.1	− 54.6	− .88	7,500
3	13,630	+ .96	+13,085	28.1	+ 22.9	+ .37	6,800
4	13,630	+ .68	+ 9,268	49.5	+ 28.6	+ .45	7,700
5	12,930	+ .89	+11,508	33.3	+ 23.9	+ .38	8,800
6	11,760	+ .65	+ 7,644	38.1	+ 18.2	+ .29	8,600
7	10,360	− .67	− 6,941	43.9	− 19.0	− .30	7,800
8 p.m.	8,720	− .82	− 7,150	9.6	− 4.3	− .07	9,200
¼ depth:							
7 a.m.	×	—	—	—	—	—	—
8	10,080	−1.79	−18,043	57.9	− 65.2	−1.04	7,200
9	10,080	−2.04	−20,563	79.7	−102.3	−1.64	7,500
10	10,080	−2.03	−20,462	55.0	− 70.2	−1.13	7,500
11	10,080	−1.75	−17,640	35.1	− 38.6	− .62	5,800
12	10,080	− .95	− 9,576	65.6	− 39.2	− .63	6,600
1	10,080	− .54	− 5,443	34.4	− 11.7	− .19	6,100
2	10,080	+ .82	+ 8,266	33.9	+ 17.5	+ .28	8,000
3	10,080	+1.13	+11,390	41.7	+ 29.6	+ .47	7,500
4	10,080	+1.75	+17,640	31.8	+ 35.0	+ .56	10,400
5	10,080	+1.55	+15,624	26.2	+ 25.5	+ .41	10,900
6	10,080	+ .70	+ 7,056	10.3	+ 4.5	+ .07	7,500
7	10,080	− .62	− 6,250	31.3	− 12.2	− .20	8,300
8 p.m.	10,080	− .98	− 9,878	25.6	− 15.8	− .25	9,200
½ depth:							
7 a.m.	×	—	—	—	—	—	—
8	10,000	−1.75	−17,500	4.6	− 5.0	− .08	7,800
9	10,000	−2.74	−27,400	26.2	− 44.8	− .74	7,500
10	10,000	−2.41	−24,100	47.0	− 70.7	−1.13	7,000
11	10,000	− .63	− 6,300				
12	10,000	− .52	− 5,200	23.3	− 7.6	− .12	6,000
1	10,000	+ .33	+ 3,300	45.3	+ 9.3	+ .15	5,900
2	10,000	+1.98	+19,800	23.9	+ 29.5	+ .47	6,500
3	10,000	+2.72	+27,200	38.0	+ 64.5	+1.03	9,800
4	10,000	+2.91	+29,100	48.9	+ 88.8	+1.42	16,500
5	10,000	+2.15	+21,500	61.2	+ 82.1	+1.32	16,200
6	10,000	+1.18	+11,800	41.7	+ 30.7	+ .49	9,500
7	10,000	+ .56	+ 5,600	60.2	+ 21.0	+ .34	8,500
8 p.m.	10,000	− .54	− 5,400	37.1	− 12.5	− .20	9,700
¾ depth:							
8 a.m.	7,800	−2.32	−18,096	42.9	− 48.6	− .78	7,200
9	7,800	−2.43	−18,954	28.8	− 34.1	− .55	7,800
10	7,800	−1.97	−15,366	70.5	− 67.7	−1.08	7,400
11	7,800	− .85	− 6,630	53.8	− 22.3	− .36	12,500
12	7,800	+ .41	+ 3,198	67.3	+ 13.4	+ .22	6,500
1	7,800	+1.49	+11,622	25.2	+ 18.3	+ .29	6,200
2	7,800	+1.81	+14,118	39.1	+ 34.5	+ .55	14,300
3	7,800	+1.65	+12,870	62.2	+ 50.0	+ .80	14,200
4	7,800	+1.63	+12,714	8.3	+ 6.6	+ .11	17,100
5	7,800	+1.30	+10,140	34.1	+ 21.6	+ .35	18,000
6	7,800	+1.29	+10,062				
7	7,800	+ .51	+ 3,978	39.7	+ 9.9	+ .16	2,800
8 p.m.	7,800	− .81	− 6,318	28.7	− 11.3	− .18	8,100
2 feet from bottom:							
8 a.m.	5,760	−1.73	− 9,964	133.3	− 83.0	−1.33	14,300
9	5,760	−1.82	−10,483	45.3	− 29.7	− .47	8,000
10	5,760	−1.36	− 7,834	15.8	− 7.7	− .12	6,800

TABLE 5.—*Typical prototype observations and computations of salinity, velocity, and sediments, Lower Hudson River at range B, station 1, April 23, 1957 —Continued*

Current meter depth (Ft.)	Weighted area	Velocity	Q	Sediment concentration	Sediment discharge		Salinity
	Ft.²	*Ft./sec.*	*C.f.s.*	*P.p.m.*	*Lb./sec.*	*C.f.s.*	*P.p.m.*
11	5,760	− .48	− 2,764	81.3	− 14.0	− .23	8,700
12	5,760	+ .51	+ 2,938	91.6	+ 16.8	+ .27	1,200
1	5,760	+1.44	+ 8,294	14.7	+ 7.6	+ .12	6,100
2	5,760	+1.33	+ 7,661	48.2	+ 23.1	+ .37	2,300
3	5,760	+1.20	+ 6,912	100.8	+ 43.5	+ .70	15,600
4	5,760	+ .75	+ 4,320	38.7	+ 10.4	+ .17	12,700
5	5,760	+ .81	+ 4,666	26.3	+ 7.7	+ .12	15,600
6	5,760	+ .59	+ 3,398	47.1	+ 10.0	+ .16	8,200
7	5,760	+ .45	+ 2,592	59.5	+ 9.6	+ .15	8,000
8 p.m.	5,760	+ .62	− 3,571	17.5	− 3.9	− .06	9,100

[1] − = downstream flow; + = upstream flow.

multiplying the sediment concentration in parts per million, as determined from the analysis of the water samples and by the velocity and area for applicable parts of the cross-sectional area (table 5 and figures 9, 13 through 19).

The net sediment discharge results are shown in table 4. The net total flows of sediment are shown on figure 20 for each station in each range in order to demonstrate the apparent circulation of the sediment in the Lower Hudson River. The plot indicates that at the upstream ranges D and E, the net flow on the New York side is upstream (stations 2 and 3, ranges D and E, fig. 20). Because of the constriction at the George Washington Bridge, the net flow and much of the sediment do not pass upstream but change direction in the region of range E and flow downstream on the New Jersey side (station 3, ranges D and E, fig. 20). On the New York side in the downstream ranges A, B, and C, the net flow is downstream. It therefore can be presumed that scour is occurring on the New York side in the upstream half of the river and that, in part, it accounts for the natural deep water found here.

On the New Jersey side, in the upstream half of the river along the 30-foot navigable channels, the flow is downstream except at the downstream point of the 30-foot channel, where the net flow is upstream (station 1, range C, fig. 20). Therefore, deposition should be expected and does occur here, as evidenced by the location of shoal 4, an area of heavy shoaling in the Lower Hudson River.

If the total discharge of sediment for the tidal cycle at each range is taken as the net amount in the indicated direction, the net volume of materials moving up or down the river would follow:

At range B, — 30.12 c.f.s. would represent 48,300 cubic yards per tidal cycle, or about 34 million cubic yards annually moving downstream.

At range C, — 6.90 c.f.s. would represent 11,070 cubic yards per tidal cycle, or about 7.7 million cubic yards annually moving downstream.

At range D, +4.83 c.f.s. would represent 7,800 cubic yards per tidal cycle, or about 5.4 million cubic yards annually going upstream.

At range E, — 3.36 c.f.s. would represent 5,400 cubic yards per tidal cycle, or about 3.8 million cubic yards annually going downstream.

As to the differentiation of net flow between upper and lower layers of the river, the measured results indicate that there is a downstream (—) predominance at the top and an upstream (+) predominance at the bottom (figs. 13, 14, 16, and 17). Where the boundary between the saline density layers could be distinguished, there appeared to be greater sediment concentration in the saline layer just below the boundary (table 5).

Model Measurements

The movement of material in a tidal waterway leading to the formation of a shoal is not the simple process of a single deposit of material in a given location but the end result of a complicated series of pickups and deposits and movements to and fro. This action is complex as to the varied interaction of many forces in the limited understanding of the many aspects of tidal hydraulics. Under such circumstances, the model is a necessary and effective tool to supplement prototype measurements in the study of this problem, because it effectively integrates the many forces and influences.[2]

Models

Two models were constructed and tested at the Waterways Experiment Station; one a comprehensive model of the Lower Hudson River with all the adjacent channels that affect the tidal regimen, and the other a larger scale model of the Lower Hudson River from the Battery to the George Washington Bridge (figs. 21, 22, and 23). The comprehensive model was

[2] DOUMA, J. H. ENGINEERING PROBLEMS IN U.S. TIDAL WATERWAYS. Amer. Soc. Civil Engin. Paper 789. 1955.

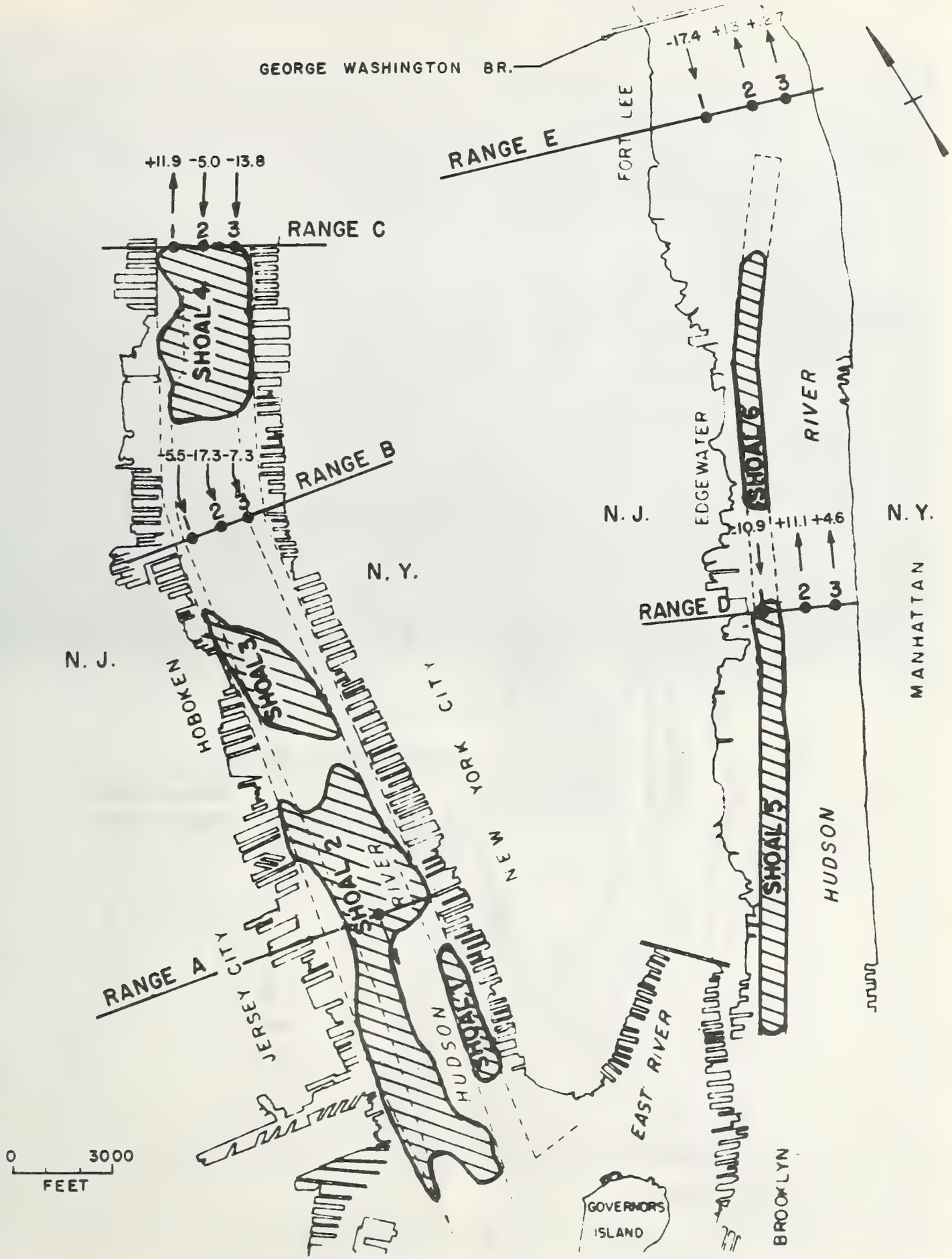


FIGURE 20. — Location of prototype sediment ranges.

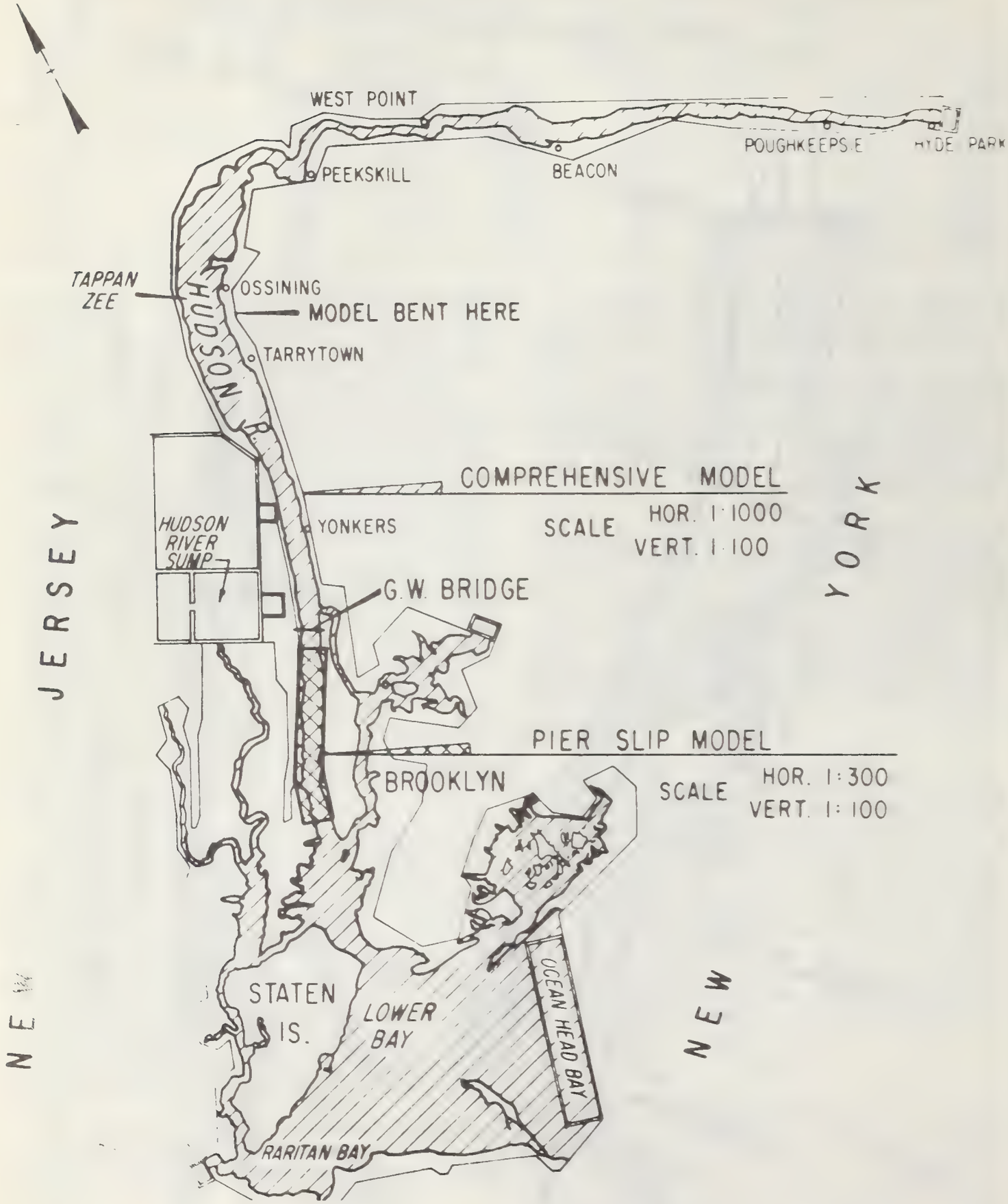


FIGURE 21. — Lower Hudson River model layouts.

FIGURE 22. — Aerial photograph of lower New York Harbor.

used to test the effectiveness of several different types of proposed channel improvement plans, in decreasing the shoaling in the channel and slips. The larger scale model of the limited area was used to test the effectiveness of certain proposed slip improvement plans for decreasing the shoaling of the slips.

The comprehensive model is quantitatively accurate as to the reduction of the tides, the tidal currents, salinity distribution, and the complex relations of bottom and top currents that occur in tidal estuaries as a result of the interplay of salt water and fresh water. The section model is quantitatively accurate with respect to hydraulic conditions within the pier slips. Fresh water was used in the section model, since density current phenomena do not affect the pier slip shoaling problem in the same degree as the channel shoaling. The techniques used to reproduce the shoaling tendencies in the models are considered qualitatively accurate and to be susceptible of reasonably quantitative interpretation. This statement is based upon the facts that (1) the same technique (of using gilsonite as a model shoal material) has been confirmed to be accurate in models of many estuaries, and (2) an accurate verification of the Lower Hudson River models was achieved. It is also considered that the model shoaling tendencies are accurate with respect to the prototype, regardless of the lack of precise knowledge as to the source of all of the material ending up as shoals in the channels and pier slips. The reason for this statement lies in the basic hydraulic mechanism of the harbor as previously described.

For the comprehensive model, a base test was established by conducting a shoaling test with the existing waterway so as to produce measured annual conditions of shoaling in nature, identified as shoal areas 1 to 6, fig. 6. The material was then retrieved from the channel and slips and measured as the base index.

The comprehensive model base tests were conducted at various fresh water flows ranging from 3,000 to 40,000 cubic feet per second. A flow of 12,000 cubic feet per second, which is approximately the median flow of the Lower Hudson River, was used as the base for all tests unless otherwise noted. An identical test was conducted with the improvements in place, and the material was retrieved from the waterway bottom. The effectiveness of each plan was

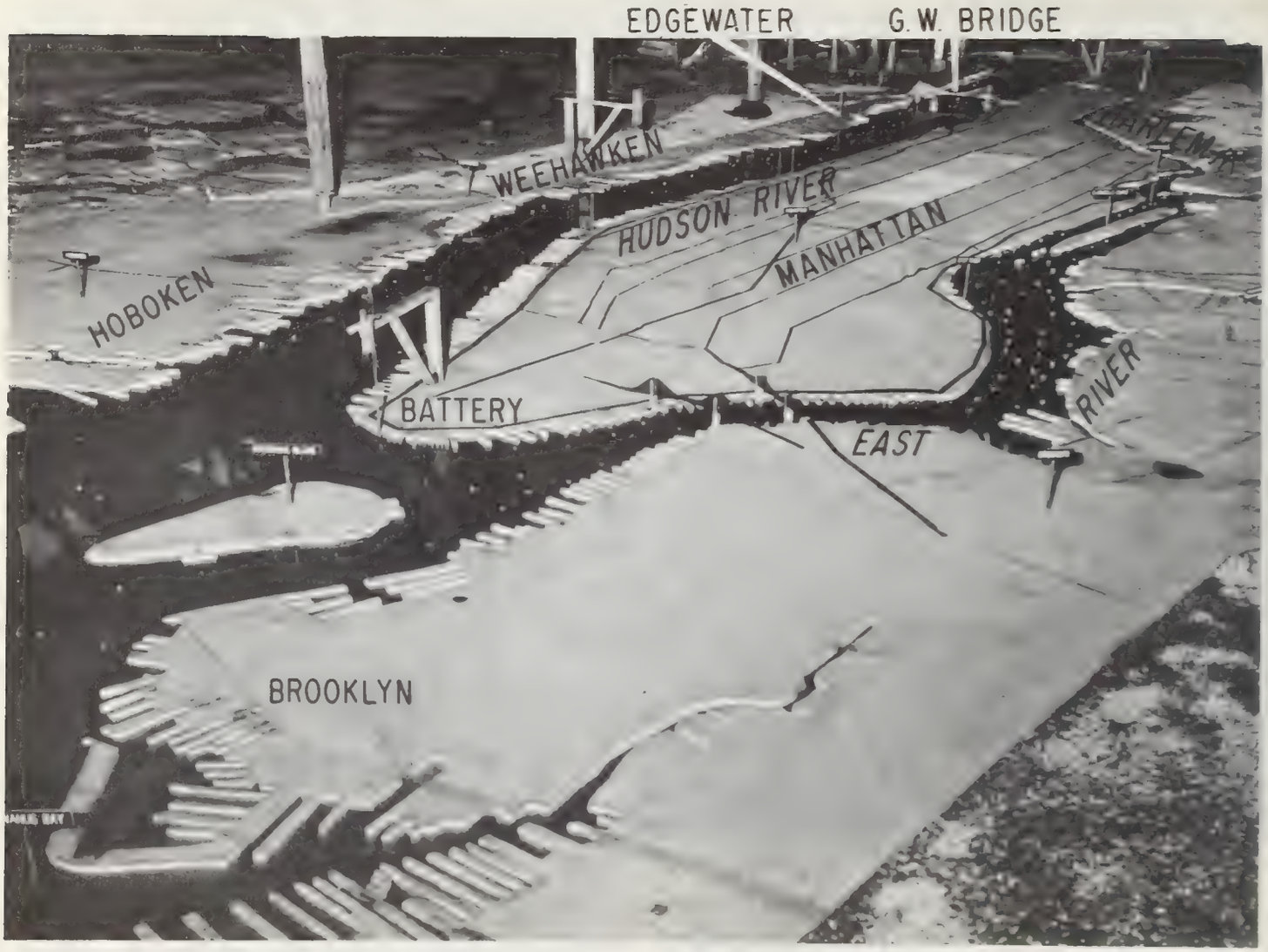


FIGURE 23.—Photograph of comprehensive model, lower New York Harbor.

evaluated on the basis of the reduction or increase in the amount of material retrieved from the channel and slips. This was recorded as a shoaling index, which is the ratio of the volume of material shoaled during the base test to the volume shoaled as a result of the plan of improvement. An index of unity indicates no change after improvement, an index less than unity indicates a reduction in shoaling, and an index greater than unity indicates an increase in shoaling. The quantity of shoaling as a result of each plan is the product of the prototype volume and the index.

Model Investigations

The basic philosophy in the reduction of dredging costs in the Lower Hudson River channels is that it could be effected in three ways, alone or in combination. The first of these would involve effecting physical changes that would alter the hydraulic mechanism so that sediments, in whole or in part, would be carried and deposited outside of the navigation facilities. The second would involve reduction of sediment entering the Lower Hudson River. The third would be the deposition of the sediment in such a way that it could be removed with a reduction in dredging costs. The plans tested in the models would fall under the first and third method.

In the comprehensive model 28 plans or variations thereof have been tested to date (table 6). Twelve of these plans involve sedimentation basins at various locations and depths (fig. 24). Two plans involve the closure of the Harlem River (fig. 25). Two plans involve the deepening of the shallow channel areas in the Tappan Zee and Haverstraw Bay Areas (fig. 26). One plan provides for a wing dike enclosing the 30-foot channel (fig. 27). Two plans provide for restoration of the pre-1941 channel cross section at the George Washington Bridge for comparison with present conditions and for determining the effect on shoaling during the period

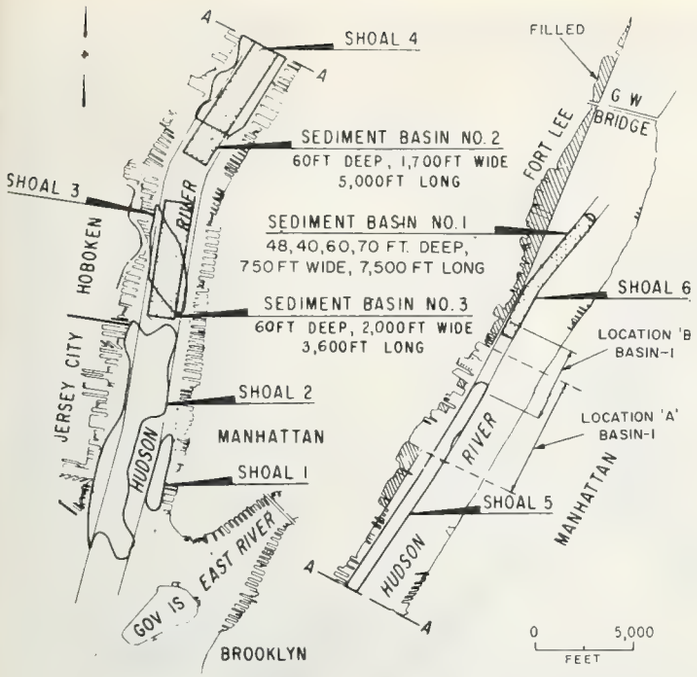


FIGURE 24. — Comprehensive model of basin series.

where dumping occurred at the bridge between 1941 and 1946.

Five plans provide for modifying the cross sections of the river at George Washington Bridge, with and without a dike parallel to the New Jersey shore (figs. 28 and 29). Four plans provide for enlarging and deepening to 50 feet the 30-foot project channel along the west, New Jersey side, and filling or constructing submerged dikes on the east, New York side of the river (fig. 30).

In the sectional (pier slip) model, 11 plans have been tested as listed in table 7 and shown on figure 31. Nine of these plans considered methods of reducing slip shoaling by physical changes to affect the shoaling mechanism by use of submerged dikes and air and water screens. Two plans considered the reduction in cost of dredging slip shoals by spoiling the material from slips into the navigable channel for removal by Federal dredging operations and by measuring the amount of material redistributed over the channels and returned to the slips during the dredging procedure.

Model Results

The results of model investigations expressed in terms of shoaling indices are summarized in tables 8 through 11 for the comprehensive model and in tables 12 and 13 for the sectional model.

The results on sediment basin indicate an increase in deposition in the basin area and a reduction in deposition in adjacent channel areas and slips previously subjected to heavy shoaling if the basin is placed in the appropriate location. The most effective basin tested was sediment basin No. 1, located at the heaviest point of shoaling, shoal 6, which is at the upstream end of the 30-foot project (fig. 24). Some plans, table 8, showed reduction in channel shoaling of more than 50 percent and in slip shoaling of 20 percent. However, plans designed for reduction of channel shoaling are not to be expected to reduce slip shoaling by more than 20 to 25 percent, therefore, a major shoaling problem would still exist in the slips after the best possible plan of channel improvement has been effected.

The complete closure of the Harlem River (fig. 25) during ebbtide discharge into the Hudson River showed significant effect in reducing shoaling in the channels by 40 percent and in slips by 32 percent (table 9). However, from the standpoint of navigation, a closure of the river would require provisions to permit the maintenance of navigation.

The deeper channels in Haverstraw Bay (fig. 26) were ineffective in reduction of shoaling (table 9) because of the local interruption of upstream predominance at the bottom flow downstream of the George Washington Bridge. These tests would probably be more effective if the local interruption of upstream predominant bottom flow could be eliminated by corrective action at the bridge and closure of the Harlem River.

Several plans of compensating dredging to approach the 1941 section at the George Washington Bridge (figs. 28 and 29) and to reduce local turbulence, showed a substantial reduction in channel and slips shoaling up to 40 percent (table 10). The variation of fresh water flow with a plan involving only compensating dredging at the bridge (plan 27) resulted in proportional variation of the shoaling (fig. 29 and table 11). Several plans involving compensating dredging at the bridge and realinement of a deep channel on the New Jersey side and a shallow channel on the New York side showed some promise (fig. 30 and table 10). These tests show that a proper enlargement of cross section at the bridge encourages upstream predominant bottom currents to carry sediment upstream of the bridge. The realinement of deep channel from the New York to the New Jersey side with construction of a sill or fill in the naturally deep water on the New York side would encourage upstream predominant currents in the deepened New Jersey channel so as to move sediment progressively upstream of the bridge.

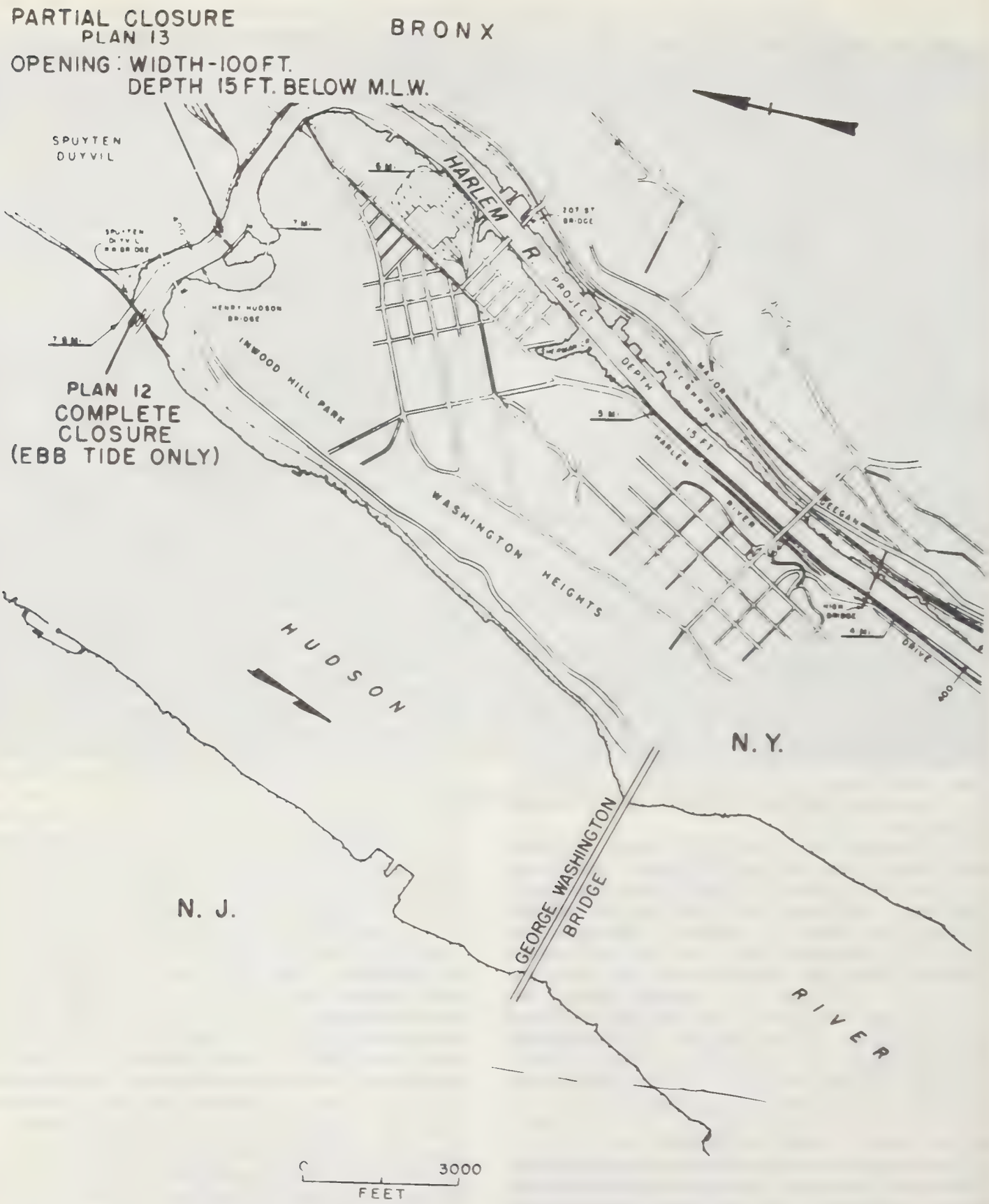


FIGURE 25. — Comprehensive model of Harlem River series.

TABLE 6.—*Lower Hudson River, comprehensive model, channel improvement plans*

SEDIMENTATION BASIN SERIES (fig. 24)

Plan No.	Basin No.	Length	Width	Depth	Capacity	Remarks
		Ft.	*Ft.*	*Ft.*	*Million cubic yards*	
1	1	7,500	750	48	4.0	
2	2	5,000	1,750	60	4.0	
3	3	3,600	2,000	60	4.0	
4	1 and 2				8.0	
5	1, loc A	7,000	750	70	9.1	End slopes 1 on ½.
6	1, loc A	7,000	750	70	9.4	End slopes 1 on 5.
7	1, loc C	4,350	750	70	5.7	End slopes 1 on 1.
8	1	7,500	750	40	2.2	
9	1	7,500	750	55	5.8	
10	1	7,500	750	70	9.8	Varying depth.
11	1	7,500	750	70	9.8	Fill to U.S. pier and bulkhead line.
28	1D	7,000	750	70	9.2	Divided at pipeline crossing.
29	1D	7,000	750	50	4.2	

HARLEM RIVER SERIES (fig. 25)

Plan No.	Description
12	Complete closure for Harlem River Ebbflow.
13	Partial closure, opening: width—100 feet; depth −15 feet mean low water.

HAVERSTRAW BAY SERIES (fig. 26)

14	Channel, Tappan Zee and Haverstraw Bay, −35 feet mean low water, 600 feet wide.
15	Channel, Tappan Zee and Haverstraw Bay, −40 feet mean low water, 1,000 feet wide.

WING DIKE SERIES (fig. 27)

16	Dike: 13,600 feet long, top elevation at mean high water from Fort Lee to and along 30-foot project.

GEORGE WASHINGTON BRIDGE SERIES (figs. 28 and 29)

17	Restoration of 1941 section prior to spoil dumping; excavation 7.5 million cubic yards.
18	Plan 17 with spoil simulation, 1942–45.
19	Compensatory dredging to restore 1941 section with channel cut; 6,800 feet long, 200 to 1,800 feet wide, 40 to 70 feet below mean low water, 5.0 million cubic yards.
20	Adjusted cross section: 10,200-foot dike, Fort Lee to Edgewater; channel 16,400 feet long, 500 to 1,100 feet wide, 57 to 60 feet deep; dredge 10.8 million cubic yards.
21	Adjusted cross section: 10,500-foot dike, Fort Lee to Edgewater; channel 10,000 feet long, 500 to 1,300 feet wide, 55 to 70 feet deep; dredge 7.1 million cubic yards.
22	Adjusted cross section: 10,725-foot dike, Fort Lee to Edgewater; channel equal to plan 21.
27	Similar to plans 20 and 21 without dike.

REALINED CHANNEL SERIES (fig. 30)

23	Deepen 30-foot channel to 50 feet below mean low water, widen to 1,000 feet, and extend to 900 feet upstream of George Washington Bridge; construct 13 submerged sills on New York side with top elevation 20 to 40 feet below mean low water.
24	Same as plan 23 except place solid fill to 30 feet below mean low water on New York side.
25	Same as plan 23 except extend 50-foot channel to 4,500 feet upstream of the bridge with maximum depths to 63 feet below mean low water and width of 1,400 feet; only a single 1,500-foot dike across New York side at West 134th Street, with top elevation at 30 feet below mean low water.
26	Same as plan 25 on New Jersey side and plan 24 on New York side.

Several plans tried in the section model (fig. 31 and table 12) showed that submerged wing dikes were ineffective, that submerged movable sills at appropriate elevations across the entrance of the slips would reduce shoaling appreciably, that an air bubble screen across the entrance or longitudinally in the slip would result in appreciable reduction of shoaling, and that a water bubble screen was ineffective. Two other tests (table 13) that showed promise were those permitting the dumping of slip spoil into the channels to be removed by Federal dredging at local cost.

Because of the inconsistency of some results and the promising results shown by several plans to effect reduced channel and slip shoaling, a limited additional test program is scheduled to permit the economic selection of the most promising plan to reduce the shoaling in the Lower Hudson River.

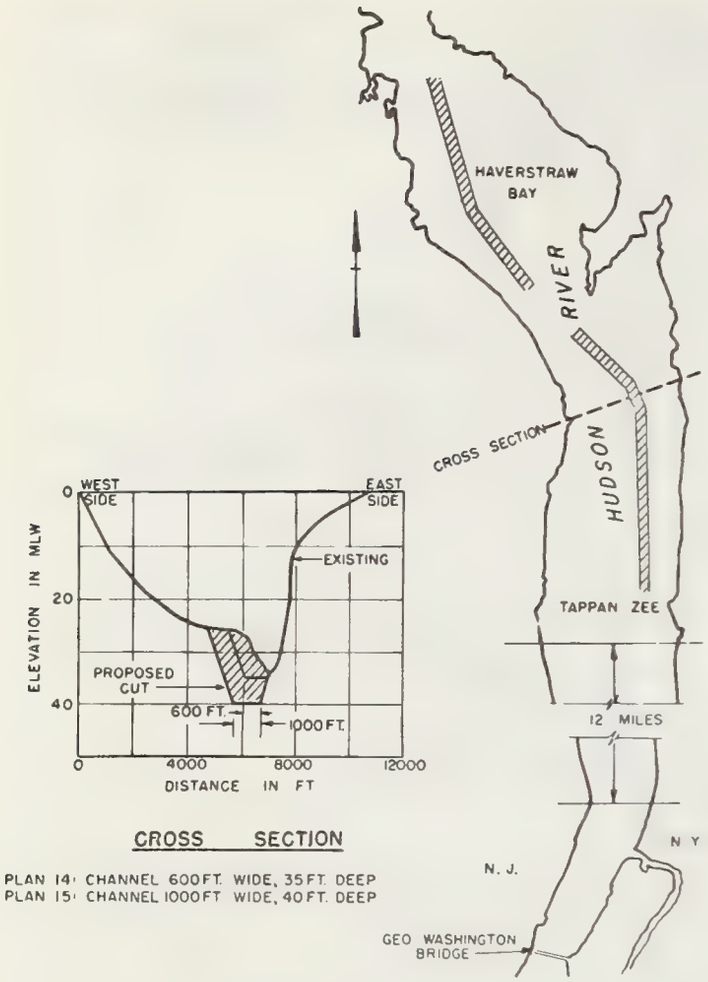


FIGURE 26. — Comprehensive model of Haverstraw Bay series.

Summary

The data presented point to following pertinent facts:

1. The Lower Hudson River is a partly mixed estuary that is a part of a complex estuarine system of the New York Harbor. This system does not lend itself to mathematical analysis and requires the use of the model tool to integrate the many influencing forces and factors.

2. The model studies have permitted a more complete understanding of the hydraulic and sediment mechanism affecting shoaling. This hydraulic and sediment mechanism is confirmed by the limited number of prototype measurements of salinity, velocities, and sediment concentration, even though variable. This better understanding is another tool to evaluate spoil and dredging operations that could affect the mechanism that results in adverse shoaling conditions.

3. The Hudson River Watershed and areas tributary to the New York Harbor are definite sources of shoal material. The part of the source that is susceptible to reduction would have little effect on the shoaling pattern.

4. These model studies have demonstrated ways of effecting reduction in channel and slip shoaling that are entwined with economics of cost of plan vs. benefits derived. The testing program cannot be predicted and fixed but must be fluid, so that it can be developed in phase with the results of the testing. Therefore, the testing program should be expanded as required until all the necessary factors have been developed. The results of the tests must be carefully examined and rechecked to assure the reliability of results.

5. The combination of limited prototype measurements supplemented by appropriate model study and analysis of results is a strong and economical tool in the study and understanding of estuary sediment problems that can lead to possible methods of solution.

Acknowledgments

The material presented herein is the result of studies conducted by the U.S. Army Engineer District, New York, in cooperation with U.S.

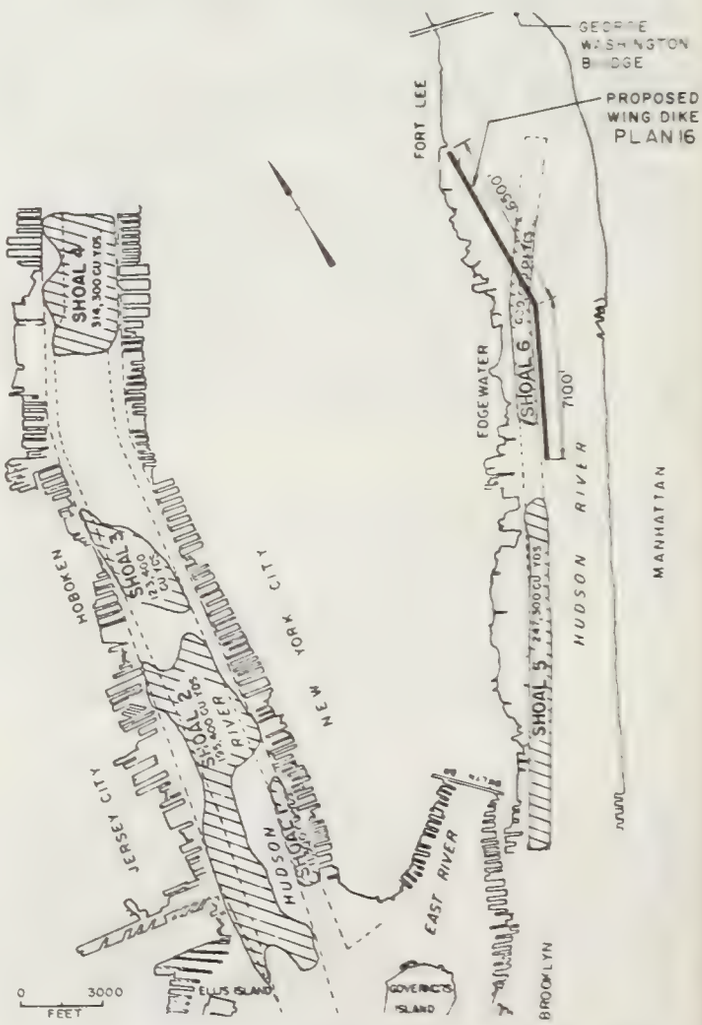


FIGURE 27. — Comprehensive model of Wing dike.

TABLE 7.—*Lower Hudson River sectional model pier slip improvement plans (fig. 31)*

Plan No.	Description
1....	Submerged dike across slip entrance, with top elevation at 35 feet below mean low water.
2....	Same as plan 1, with top elevation at 5 feet below mean low water.
3....	Wind Dike: 100-foot extension of pier 92, 20 feet deep, with top elevation at mean low water.
4....	Same as plan 3, except top elevation at 35 feet below mean low water.
5....	Same as plan 3, except dike extension of pier 90.
6....	Submerged air screen across slip entrance, with top elevation of conduit at 45 feet below mean low water (discharge, 37,500 c.f.m.).
7....	Same as plan 6, except water screen (discharge, 18,000 c.f.m.).
8....	Air bubble turbulence generator consisting of two parallel, longitudinal pipes at ⅓ points, with top elevation 45 feet below mean low water (discharge, 232,500 c.f.m.).
9....	Same as plan 8 (discharge, 30,000 c.f.m.).
10....	Simulated removing shoal in slip 90-92 by a bucket dredge filling a 1,750 cubic yard barge every 6 hours and dumping at mid-navigable channel in front of slip. Operation continued until 74,500 cubic yards (half annual shoaling) were removed.
11....	Simulated removal of same volume of shoal as in plan 10 by a hydraulic dredge discharging at the surface at 500 feet toward the channel from the face of pier 92 equal to 9 prototype days.

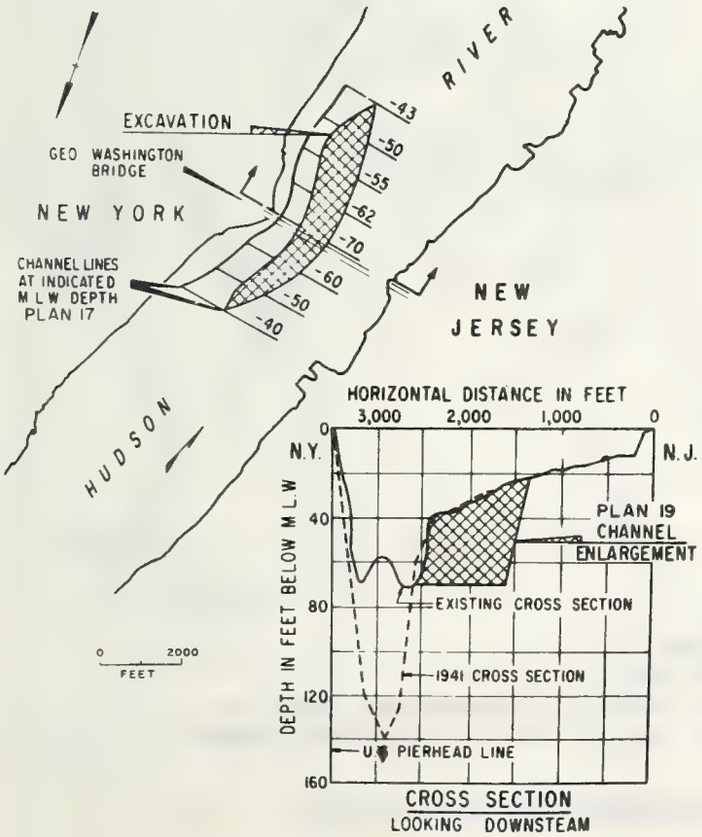


FIGURE 28. — Comprehensive dredging to 1941 cross section.

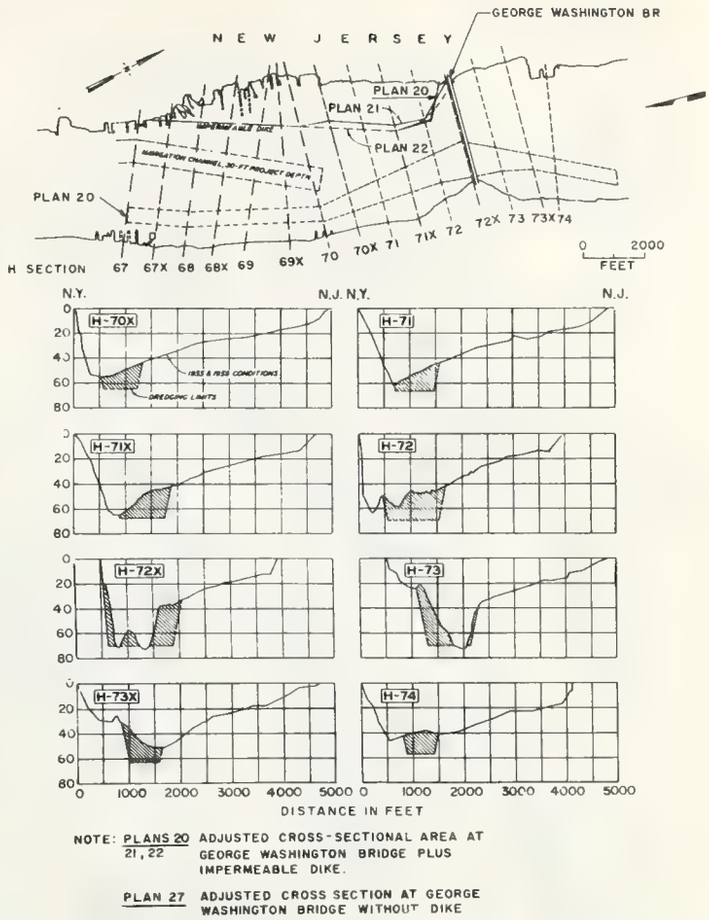


FIGURE 29. — Comprehensive model of George Washington Bridge series.

Army Engineer Waterways Experiment Station, Vicksburg, Miss. The writer acknowledges the leadership and direction of Col. Milton M. Miletich, Col. Charles M. Duke, and Brig. Gen. Thomas De F. Rogers who served as U.S. Army District Engineers, and the technical guidance of Charles K. Panish, Chief of the Engineering Division of the New York District; and Col. Alex G. Sutton, Jr. and Col. Edmund H. Lang, who served as directors, and Joseph B. Tiffany, technical director of the Waterways Experiment Station. Special acknowledgment is due to Henry B. Simmons, chief of the Estuaries Section of the Waterways Experiment Section, who was in direct charge of conducting the model studies. The writer also acknowledges the assistance in the research work and preparation of this paper of the following District personnel: Isiah Katcher, Aaron J. Addis, Melvin Mark, Charles Beatty, and Joseph Tennen.

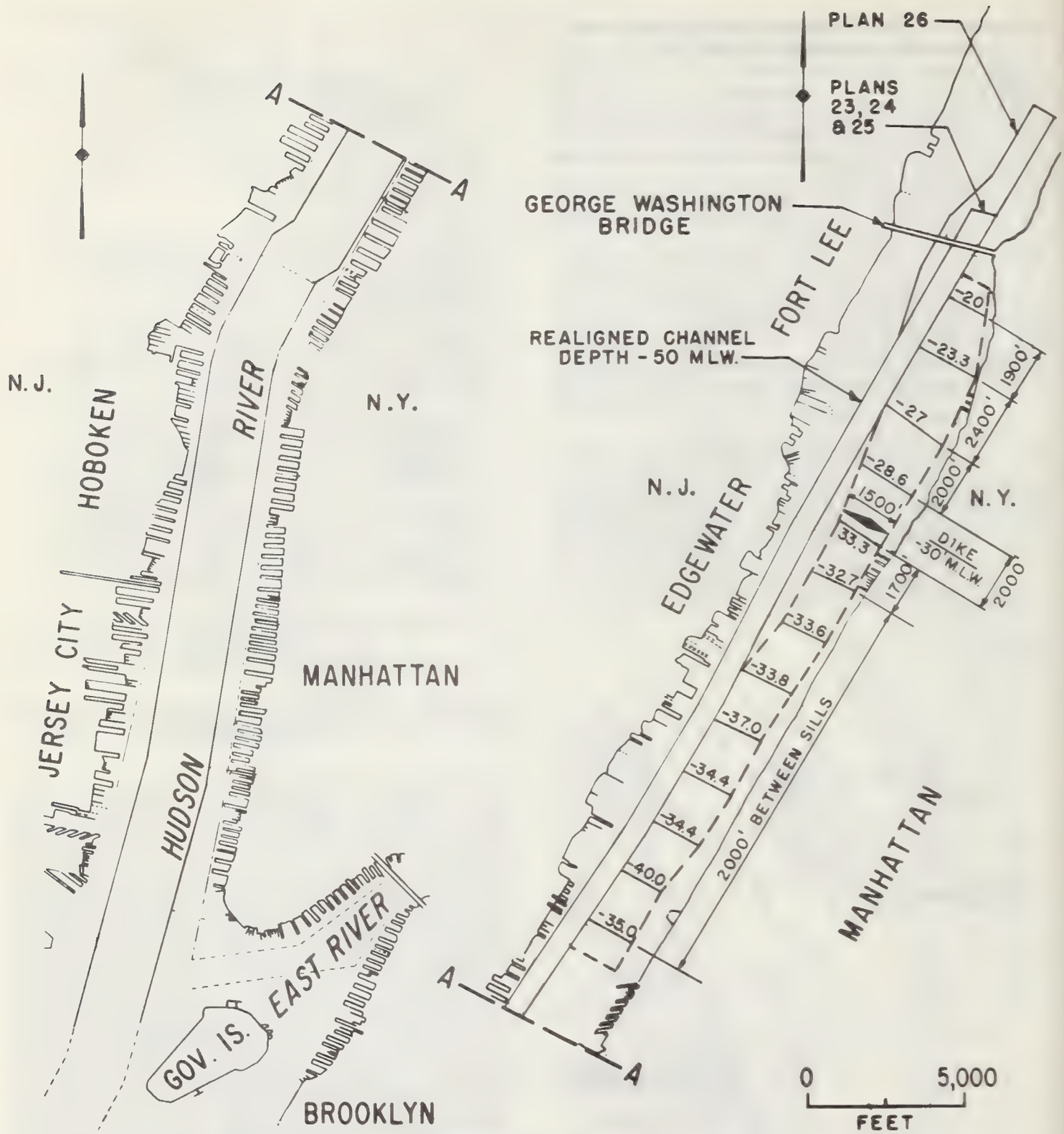


FIGURE 30. — Plan for realined channel.

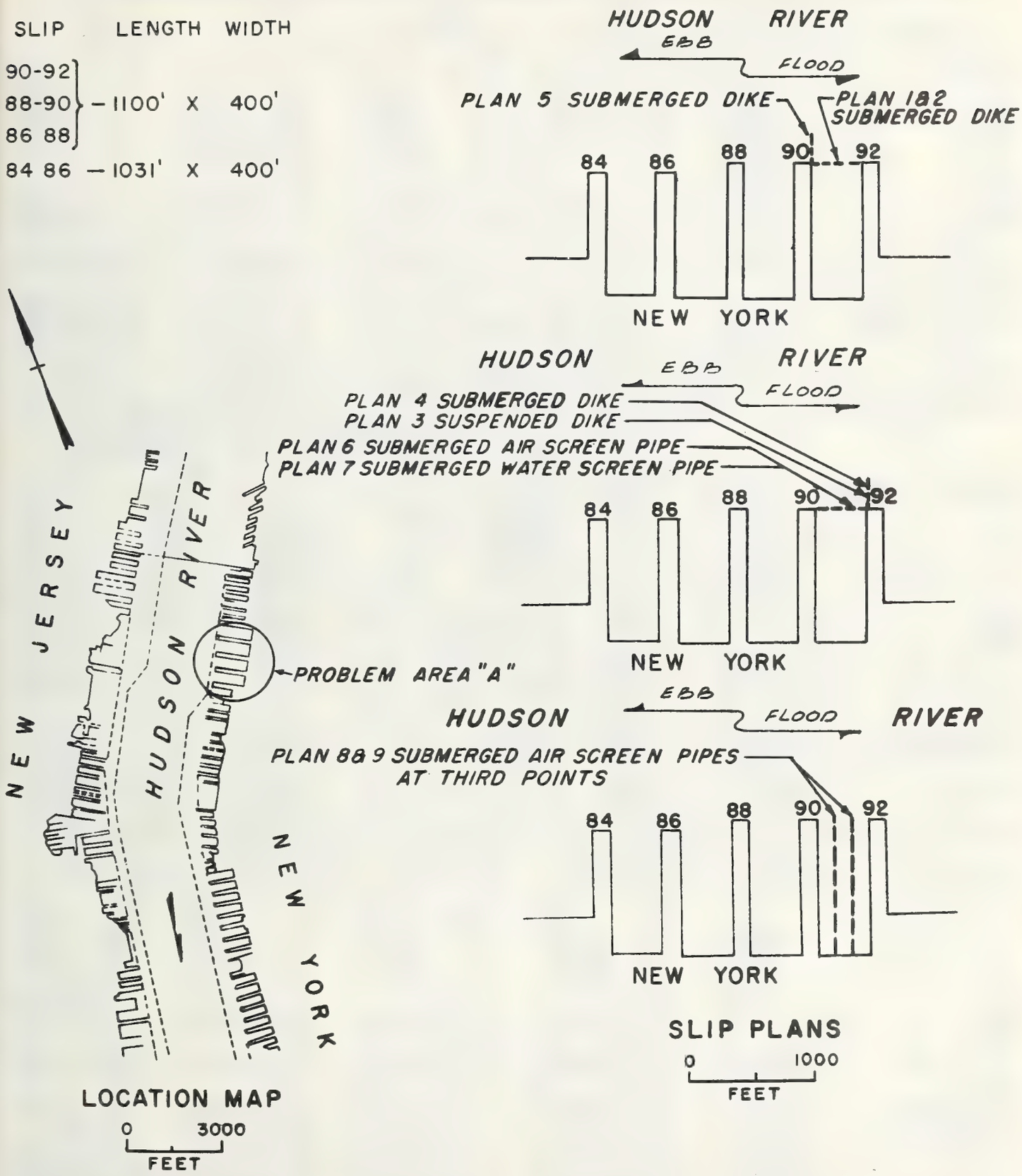


FIGURE 31. — Sectional pier slip model for shoaling studies.

TABLE 8.—*Lower Hudson River comprehensive model studies: sediment basin series*

Channel	Prototype	Plan 1, Basin No. 1		Plan 2, Basin No. 2		Plan 3, Basin No. 3		Plan 4, Basin Nos. 1 and 2	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:									
1	11,300	0.44	5,000	0.78	8.800	1.67	18,900	0.89	10,100
2	195,400	.91	177,800	1.37	267,700	.95	185,600	.92	179,800
3	123,400	.44	54,300	.76	93,800	.44	54,300	.50	61,700
4	314,300	1.07	336,300	.43	135,700	.51	160,300	.41	128,900
5	247,300	.31	76,700	.56	138,500	.83	205,300	.82	202,800
6	689,600	.....	.....	.73	503,400	.64	441,300	.....	.....
Shoals	1,581,300	.41	650,100	.73	1,147,900	.67	1,065,700	.37	583,300
Basins	.....	1.28	882,700	1.08	339,400	.64	79,000	.....	[1]1,654,900
Subtotal	1,581,300	.97	1,532,800	.94	1,487,300	.72	1,144,700	1.41	2,238,200
Slip areas:									
New York side	1,062,300	.98	1,041,100	.72	764,900	.81	860,500	.81	860,500
New Jersey side	1,937,700	.94	1,821,400	1.21	2,344,600	1.44	2,790,300	.97	1,879,600
Subtotal	3,000,000	.95	2,862,500	1.04	3,109,500	1.22	3,650,800	.91	2,740,100
Total	4,581,300	.96	4,395,300	1.00	4,596,800	1.05	4,795,500	1.09	4,978,300

Channel	Prototype	Plan 5, Basin No. 1, loc. A, End slope 1 on ½		Plan 6, Basin No. 1, loc. A, End slope 1 on 5		Plan 7, Basin No. 1, loc. C, End slope 1 on 1	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:							
1	11,300	1.50	17,000	0.75	8,500	0.75	8,500
2	195,400	1.12	218,800	1.06	207,100	1.41	275,500
3	123,400	1.08	133,300	1.17	144,400	1.83	225,800
4	314,300	.78	245,200	.89	279,700	1.40	440,000
5	247,300	.87	215,200	.67	165,700	2.33	576,200
6	689,600	.29	200,000	.08	55,200	.29	200,000
Shoals	1,581,300	.65	1,029,500	.54	860,600	1.09	1,726,000
Basins	.....	.....	1,687,000	.....	318,300	.....	530,500
Subtotal	1,581,300	1.71	2,716,500	.75	1,178,900	1.43	2,256,500
Maintained slips:							
New York side	1,062,300	.48	509,900	.86	913,600	.88	934,800
New Jersey side	1,937,700	.80	1,550,200	.93	1,802,100	1.42	2,751,500
Subtotal	3,000,000	.69	2,060,100	.91	2,715,700	1.23	3,686,300
Total	4,581,300	1.04	4,776,600	.83	3,894,600	1.30	5,942,800

Channel	Prototype	Plan 8, Basin No. 1 Depth = −40 ft.		Plan 9, Basin No. 1 Depth = −55 ft.		Plan 10, Basin No. 1 Depth = −70 ft.		Plan 11, Basin No. 1 Depth = −70 ft.	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:									
1	11,300	0.75	8,500	1.50	17,000	0.75	8,500	0.75	8,500
2	195,400	1.34	261,800	1.34	261,800	1.40	273,600	1.14	222,800
3	123,400	1.96	241,900	1.79	229,900	2.12	261,600	2.62	323,300
4	314,300	1.05	330,000	1.42	446,300	1.33	418,000	1.77	556,300
5	247,300	.83	205,300	.70	173,100	.73	180,500	1.10	272,000
6	689,600	.....	.....	.....	.....	.....	.....	.....	.....
Shoals	1,581,300	.66	1,047,500	.71	1,119,100	.72	1,142,200	.87	1,382,900
Basins	.....	3.36	2,317,100	4.31	2,972,200	5.70	3,930,700	5.46	3,765,200
Subtotal	1,581,300	2.13	3,364,600	2.58	4,991,300	3.21	5,072,900	3.25	5,148,100
Slip areas:									
New York side	1,062,300	.53	563,000	.74	786,100	.78	828,600	.62	658,600
New Jersey side	1,937,700	1.20	2,325,200	.78	1,511,400	.90	1,743,900	.68	1,317,600
Subtotal	3,000,000	.96	2,888,200	.77	2,297,500	.86	2,572,500	.66	1,976,200
Total	4,581,300	1.36	6,252,800	1.39	6,388,800	1.67	7,645,400	1.56	7,124,300

[1] Total in basins 1 and 2: basin No. 1, 1,268,900 cu. yd., index 1.84; basin No. 2, 386,000 cu. yd., index 1.23.

TABLE 8.—*Lower Hudson River comprehensive model studies: sediment basin series—Continued*

Channel	Prototype	Plan 28 Basin No. 1 (divided)		Plan 29, Basin No. 1 (divided)			
				Depth = −50 ft. m.l.w. 3,000 c.f.s. discharge		Depth = −50 ft. m.l.w. 40,000 c.f.s. discharge	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	1.36	15,400	0.40	4,500	1.88	21,200
2	195,400	1.09	213,000	.67	131,000	.51	99,700
3	123,400	.94	116,000	.42	51,800	.45	35,500
4	314,300	1.04	326,900	.65	204,300	.41	128,900
5	247,300	1.09	269,600	.50	123,700	.38	94,000
6	689,600	.....	.....	.01	6,900	.05	34,500
Shoals	1,581,300	.80	940,900	.33	522,200	.27	433,800
Basins	.....	4.30	2,965,300	5.50	[2] 3,792,800	8.65	[3] 5,965,000
Subtotal	1,581,300	2.47	3,906,200	2.73	4,315,000	4.04	6,398,800
Slip areas:							
New York side	1,062,300	.07	924,200	1.10	1,168,500	.99	1,051,700
New Jersey side	1,937,700	1.30	2,519,000	.72	1,395,100	.88	1,705,200
Subtotal	3,000,000	1.15	3,443,200	.85	2,563,600	.92	2,756,900
Total	4,581,300	1.60	7,349,400	1.50	6,878,600	2.00	9,146,700

[2] 57 percent in south part of basin; 43 percent in north part.
[3] 22 percent in south part of basin; 78 percent in north part.

TABLE 9.—*Lower Hudson River comprehensive model studies: Harlem, Haverstraw, realinement series and George Washington Bridge*

Channel	Prototype	Harlem River closure				Haverstraw Bay			
		Plan 12 Ebb closure		Plan 13 Partial closure		Plan 14 35-foot channel		Plan 15 40-foot channel	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	0.56	6,300	0.44	5,000	0.89	10,100	1.06	12,000
2	195,400	.80	156,300	.70	136,800	1.13	220,800	1.10	214,900
3	123,400	.62	76,500	.88	108,600	.78	96,300	1.63	201,100
4	314,300	1.14	358,300	.77	242,000	.96	301,700	1.29	405,400
5	247,300	.21	51,900	.79	195,400	.74	183,000	1.10	272,000
6	689,600	.42	289,600	1.14	786,100	1.09	751,700	.94	655,100
Subtotal	1,581,300	.59	938,900	.93	1,473,900	.99	1,563,600	1.11	1,760,500
Slip areas:									
New York side	1,062,300	.95	1,009,200	.76	807,300	1.23	1,306,600	.61	648,000
New Jersey side	1,937,700	.68	1,317,600	.89	1,724,600	1.45	2,809,700	.71	1,375,800
Subtotal	3,000,000	.78	2,326,800	.84	2,531,900	1.37	4,116,300	.68	2,023,800
Total	4,581,300	.71	3,265,700	.87	4,005,800	1.24	5,679,900	.83	3,784,300

Channel	Prototype	Realinement studies				Plan 25 Single sill		George Washington Bridge	
		Plan 23 With sills		Plan 24 Solid fill				Plan 26 Solid fill and changed section	
Shoal areas:									
1	11,300	1.00	11,300	0.56	6,300	0.67	7,600	0.33	3,700
2	195,400	1.00	195,400	.64	125,100	.65	127,000	.47	91,800
3	123,400	1.15	141,900	.60	74,000	.27	33,300	.31	38,300
4	314,300	1.40	440,000	.54	169,700	.19	59,700	.39	122,600
5	247,300	2.02	499,500	1.42	351,200	.83	205,300	.53	131,100
6	689,600	1.04	717,200	1.11	765,500	1.17	806,800	.40	275,800
Subtotal	1,581,300	1.27	2,005,300	.94	1,491,800	.78	1,239,700	.42	663,300
Slip areas:									
New York side	1,062,300	.65	690,500	.94	998,600	.75	796,700	.71	754,200
New Jersey side	1,937,700	.57	1,104,500	.65	1,259,500	.71	1,375,800	.64	1,240,100
Subtotal	3,000,000	.60	1,795,000	.75	2,258,100	.72	2,172,500	.66	1,994,300
Total	4,581,300	.83	3,800,300	.82	3,749,900	.74	3,412,200	.58	2,657,600

TABLE 10.—*Lower Hudson River comprehensive model studies: Dike and bridge series*

Channel	Prototype	Wing dike		George Washington Bridge series					
		Plan 16		Plan 17 1941 channel		Plan 18 [1] 1942-45 spoiling		Plan 19 Compensatory dredging	
Shoal Areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	1.89	21,400	0.56	6,300	0.80	5,000	0.44	5,000
2	195,400	1.00	195,400	1.19	232,500	1.08	251,100	.93	181,700
3	123,400	.98	120,900	1.70	209,800	.72	151,000	.63	77,700
4	314,300	1.35	424,300	1.11	348,900	1.40	488,500	.57	179,200
5	247,300	1.35	333,900	.51	126,100	2.19	276,200	.43	106,300
6	689,600	.94	648,200	.70	482,700	1.30	627,500	.48	331,000
Subtotal	1,581,300	1.10	1,744,100	.89	1,406,300	1.28	1,799,300	.56	880,900
Slip areas:									
New York side	1,062,300	1.19	1,264,100	1.35	1,434,100	1.01	1,072,900	.84	892,300
New Jersey side	1,937,700	1.39	2,693,400	.70	1,356,400	2.22	4,301,700	1.11	2,150,800
Subtotal	3,000,000	1.32	3,957,500	.93	2,790,500	1.79	5,374,600	1.01	3,043,100
Total	4,581,300	1.24	5,701,600	.92	4,196,800	1.57	7,173,900	.86	3,924,000

Channel	Prototype	Adjusted cross-sectional area at George Washington Bridge					
		Plan 20 Dike plan 1		Plan 21 Dike plan 2		Plan 22 Dike plan 3	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cy. yd.*	*Index*	*Cu. yd.*
Shoal areas:							
1	11,300	0.44	5,000	0.44	5,000	0.67	7,600
2	195,400	1.46	285,300	1.44	281,400	1.39	271,600
3	123,400	1.66	204,800	1.05	129,600	1.48	182,600
4	314,300	1.20	377,200	1.09	342,600	1.23	386,600
5	247,300	.50	123,700	.74	183,000	.92	227,500
6	689,600	.59	406,900	.62	427,600	.37	255,200
Subtotal	1,581,300	.89	1,402,900	.87	1,369,200	.84	1,331,100
Maintained slips:							
New York side	1,062,300					.62	658,600
New Jersey side	1,937,700					.63	1,220,800
Subtotal	3,000,000					.63	1,879,400
Total	4,581,300					.70	3,210,500

[1] Plan 17 quantities used as base quantities for computing indices for plan 18.

TABLE 11.—*Lower Hudson River comprehensive model studies: Variable fresh water flow series*

Channel [1]	Prototype	George Washington Bridge series					
		Plan 27: no dike FWD [2] =3,000 c.f.s.		Plan 27: no dike FWD [2] =12,000 c.f.s.		Plan 27: no dike FWD [2] =40,000 c.f.s.	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	0.86	8,700	0.09	1,000	3.12	35,300
2	195,400	.95	186,600	.84	164,100	.62	121,100
3	123,400	.75	92,600	1.37	169,100	.58	71,600
4	314,300	.79	248,300	1.35	424,300	.55	172,900
5	247,300	.42	103,900	.59	145,900	.31	76,700
6	689,600	.69	475,800	.61	420,700	1.42	979,200
Subtotal	1,581,300	.71	1,115,900	.84	1,325,100	.92	1,456,800
Slip areas:							
New York side	1,062,300	.84	892,300	.90	956,100	.67	711,700
New Jersey side	1,037,700	.61	1,182,000	.78	1,511,400	.56	1,085,100
Subtotal	3,000,000	.69	2,074,300	.82	2,467,500	.60	1,796,800
Total	4,581,300	.70	3,190,200	.83	3,792,600	.71	3,253,600

Channel [1]	Base test series					
	Low flow: Hudson River discharge = 3,000 c.f.s.		*Median flow:* Hudson River discharge = 12,000 c.f.s.		*High flow:* Hudson River discharge = 40,000 c.f.s.	
Shoal areas:	*Vol. in cc.*	*Index*	*Vol. in cc.*	*Index*	*Vol. in cc.*	*Index*
1	50	2.27	22	1.0	32	1.45
2	1,165	1.82	640	1.0	816	1.27
3	480	1.92	250	1.0	816	3.26
4	1,305	1.55	840	1.0	2,614	3.11
5	120	.71	170	1.0	786	4.62
6	360	.49	740	1.0	489	.66
Subtotal	3,480	1.31	2,662	1.0	5,553	2.09
Slip areas:						
New York side	345	.86	400	1.0	416	1.04
New Jersey side	180	1.33	135	1.0	68	.50
Subtotal	525	.98	535	1.0	484	.90
Total:						
Maintained areas	4,005	1.25	3,197	1.0	6,037	1.89
Problem areas	8,309	1.14	7,263	1.0	15,839	2.18

[1] These totals for channel areas do not include maintenance of the plan channel in the vicinity of the bridge.

[2] FWD = fresh water discharge.

TABLE 12.—*Lower Hudson River sectional (pier slip) model studies: slip modification series*

Slip No.	Prototype shoaling	Plan 1 Entrance sill to −35 ft. m.l.w.		Plan 2 Entrance sill to −5 ft. m.l.w.		Plan 3 Suspended dike North side slip 90–92		Plan 4 Submerged dike North side slip 90–92		Plan 5 Submerged dike South side slip 90–92	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
84–S	13,740	0.91	12,500	1.60	22,000	1.41	19,400	1.66	22,800	1.76	24,200
84–86	67,458	1.00	67,500	1.34	90,400	1.25	84,300	1.29	87,000	1.40	94,400
86–88	50,474	1.10	55,500	1.65	83,300	1.70	85,800	2.12	107,000	1.87	94,400
88–90	80,518	.82	66,000	1.21	97,500	1.33	107,100	1.59	128,000	1.57	126,400
90–92	149,301	.64	95,600	.49	73,200	1.25	186,600	1.71	255,300	1.68	250,900
92–N	17,312	1.06	18,400	1.37	23,700	1.90	32,900	1.74	30,100	2.14	37,000
Total	378,803	.83	315,500	1.03	390,100	1.36	516,100	1.66	630,200	1.65	627,300

Slip No.	Prototype shoaling	Plan 6 Entrance air screen 37,500 c.f.m. at +6.5 p.s.i.		Plan 7 Entrance water screen 300 c.f.s.		Plan 8 Longitudinal air jet 232,500 c.f.m. at +6.5 p.s.i.		Plan 9 Longitudinal air jet 30,000 c.f.m. at +2.6 p.s.i.	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
84–S	13,740	1.52	20,900	1.20	16,500	1.30	17,900	1.20	16,500
84–86	67,458	1.22	82,300	1.16	78,300	1.39	93,800	1.37	92,400
86–88	50,474	1.62	81,800	.89	44,900	1.39	70,200	1.40	70,700
88–90	80,518	1.33	107,100	.92	74,100	1.40	112,700	1.35	108,700
90–92	149,301	.84	125,400	1.08	161,200	.04	6,000	.78	116,500
92–N	17,312	1.03	17,800	1.38	23,900	1.36	23,500	1.33	23,000
Total	378,803	1.15	435,300	1.05	398,900	.85	324,100	1.13	427,800

TABLE 13.—*Lower Hudson River sectional (pier slip) model studies: effects of dredging operations on slip shoaling*

Slip No.	Volume retrieved in cubic yards: Prototype	
	Bucket dredge and barge operation	Hydraulic dredge and pipeline operation
Slips on New York Side:		
84—South	84	93
84—86	264	264
86—88	300	264
88—90	360	248
90—92	460	295
92—North	86	155
Total	1,554	1,319
83—North	36	24
112—113	24	15
113—114	26	21
Slips on New Jersey Side:		
149 (north and south)	64	[1]
151 (north and south)	115	[1]
188 (to pierhead line)	576	[1]

[1]No significant deposits found on the New Jersey side after the hydraulic dredging operation.

SEDIMENT MOVEMENT AS DEFINED BY RADIOACTIVE TRACERS: A PRELIMINARY REPORT[1]

[Paper No. 60]

By L. L. MCDOWELL, *soil scientist (chemistry), USDA Sedimentation Laboratory, Soil and Water Conservation Research Division, Agricultural Research Service*

Introduction

An investigation of utilizing radioactive tracers in laboratory and field studies of sediment transport is now in progress at the Sedimentation Laboratory. The preliminary results of this investigation are discussed herein.

The use of radioactive tracers in sediment transport investigations has gained widespread interest in recent years. Several investigations,

[1] In cooperation with the University of Mississippi and Mississippi State University.

utilizing a variety of radioisotopes, have been conducted along coasts, in estuaries, and in maritime rivers. Considerable information has resulted from these studies. The reader is referred to the Proceedings of the 20th Permanent International Navigation Congress in Baltimore, 1961, Section II, Subject V, for the details of these investigations. Included in these proceedings are articles on radioactive tracing experiments conducted in several countries.

At the USDA Sedimentation Laboratory, research is concerned with the origin and routing of sediment in agricultural watersheds. Knowledge of the processes involved in the detachment, entrainment, transport, and deposition of the soil particles is necessary for planning the control of erosional processes. Techniques that will improve the measurement and determination of sediment transport under both overland and streamflow conditions will result in the design of more effective erosion and flood prevention structures.

The measurement of total sediment discharge in terms of hydraulic parameters and sediment properties is one of the most important objectives of research in sedimentation. Tracer techniques, including both radioactive and luminescent materials, offer a unique approach to the study of discrete particle movement, and hence to the possible evaluation of many sediment transport problems.

It is not the intention to give a thorough review of the present knowledge on this subject. It is, however, pertinent to this discussion to review some of the previous studies in terms of their objectives, materials and methods, and interpretations. Such a review will aid in the application of this method to sedimentation problems encountered in agricultural areas.

Review of Literature

Preparation of Labeled Sediment

It is axiomatic that the tracer possess the same hydraulic characteristics, i.e., transport properties, as the natural sediment. Theoretically, the tracer and the natural sediment must be equivalent in terms of these chemical and physical properties: (1) Specific gravity, (2) size distribution, (3) shape, (4) surface properties, (5) hardness, and (6) chemical stability. Any drastic differences between the tracer and sediment in question will obviously lead to differential sorting or abrasion of the tracer and, hence, to erroneous results. In practice it may be necessary to deviate somewhat from the equivalence of tracer and natural sediment. These deviations will depend largely upon the individual laboratory and its access to certain equipment and facilities necessary for the preparation and measurement of the tracer material. Such deviations must, nevertheless, be borne in mind in the final interpretation of the experimental results.

A number of radioisotopes have been used in studies of sand and silt movement. Thus far, results have been largely qualitative in nature. This is not surprising, however, in view of the inherent limitations of the method. The results, nevertheless, have been highly informative. Tracers have been prepared from both natural and artificial sediments.

Irradiation of Natural Sediments

Slow neutron irradiation has been utilized in the activation of both natural and artificial sediments. The irradiation of quartz and natural sediments has been employed by Goldberg and Inman (*7*), Inman and Chamberlain (*8*), Crickmore (*2*), and Crickmore and Lean (*3*).

Goldberg and Inman found that phosphorus-32 (beta emitter, 1.70 million electron volts (Mev), half life = 14.3 days) was the principal radioisotope resulting from the slow neutron irradiation of natural quartz sand. The beta particles emitted from phosphorus-32 are readily adsorbed (penetrating ability = 0.2 inch in water), making in situ measurements impractical. Inman and Chamberlain utilized these emissions to study the movement of beach sand at the Scripp Institute of Oceanography. Surface and core samples were taken from the experimental area and the individual samples analyzed by autoradiographic technique. In this manner, it was possible to record the presence of individual irradiated grains on photo-sensitive film.

Recently, sediment transport investigations utilizing irradiated sand were conducted by Crickmore and associates at the Hydraulics Research Station in Wallingford, England. Irradiation of untreated "Arnold 26A" sand (mean size, 170 microns) yielded both gamma and beta radiation. Autoradiography tests on this sample demonstrated the nonuniformity in activity of individual grains. Grains showing high activity were identified as the minor constituents of the sand, e.g., feldspars, micas, and magnetite. The quartz grains gave only weak activities. The sediment transport properties of the minor constituents could not be expected to be the same as quartz, the major constituent of the sand. The untreated irradiated sand was, therefore, demonstrated to be unsuitable as a tracer.

Activity measurements on various density fractions of the sand indicated the presence of a number of radioisotopes with a wide range of half-lives. The middle density fraction (specific gravity 2.60 to 2.70) was analyzed 3 months after irradiation. Scandium-46 (0.89 and 1.12 Mev) was the most active constituent, plus con-

tributions from iron-59 (1.10 and 1.29 Mev) and possibly iridium-192 (0.31, 0.47, and 0.61 Mev).

Activity distributions with respect to particle size were also performed on the different density fractions of the irradiated "Arnold 26A" sand. Analyses indicated that the specific activity was inversely proportional to particle size. This was true even for the middle density fraction (specific gravity 2.64 to 2.66), which was composed almost entirely of quartz. Decay curves for the different size fractions were similar, indicating that the radioactive constituents were the same for all fractions. Furthermore, the variation of surface activity with size conformed very closely with the variation of surface area with size. This suggested that the activity was present on the surface of the particles.

The surface label of the sand presented two problems in regard to its use as a tracer: (1) Possible losses of activity under the sediment transport conditions, i.e., from solution and abrasion effects; and (2) lack of proportionality between activity and total mass of tracer for any given point during transport.

Laboratory measurements of activity losses under severe abrasion and solution conditions showed losses of 15 to 20 percent of the activity. These activity losses occurred in the first 2 hours, with no significant subsequent change.

Crickmore desired to compare the sediment transport rates of tracer sand with the original sand in a laboratory channel. Emphasis was placed on the transport rate for the total mass of sand without concern for the movement of any definite size fraction. The variation of activity with particle size complicated this type of experimentation. In other words, for in situ measurements of the sand transported, it was desirable that the total activity at any point be proportional to the total tracer present, independent of particle size. Two possible approaches were available to correct this problem: (1) Different size fractions of the tracer could be irradiated for different lengths of time to produce the same specific activity; or (2) the amount of each size fraction could be adjusted to compensate for the lack of uniformity in specific activity. The second approach was chosen, and the tracer size distribution was weighted toward the coarser fractions to offset the bias of the activity distribution. A tracer modified in this manner can be treated as though it is labeled for bulk or volume.

The transport study was made in a large laboratory flume operating at equilibrium with a steady discharge of 10 c.f.s. At equilibrium, the sand transport was 100 pounds per hour $\pm$ 5 percent as determined by weighing the collected sediment. The middle density fraction (specific gravity 2.64 to 2.66), after irradiation and grain size modification to give a volume-labeled tracer, was added to the bed at the upstream end of the flume. Bed sand was removed from a cross section of the flume to a depth corresponding to the deepest previously observed dune trough. Active sand was mixed with this material in the ratio of 1:300; the mixture was returned to the trench and carefully molded to resemble the natural dune pattern.

Movement of the tracer was determined by a scintillation detector that could be traversed along the flume. Activity at any given point was compared to the total quantity of tracer transported for different time intervals. All measurements were referred to a standard sample to allow correction for decay of the radioisotope(s). The average velocity of transport was determined as 2.00 feet per hour from the rate of movements of the centers of gravity of the distributions. Activity measurements of the injection strip after 100 hours of flume operation gave an average thickness of movement of 0.116 foot, corresponding very closely to the depth of the deepest trough below mean bed level. A volume transport of 1.18 cubic feet per hour for the sand was computed from knowledge of the average transport rate, mean depth of movement, and width of the flume. The bulk density of sand compacted in the rippled layer was measured as 90 pounds per cubic foot. The transport rate, calculated to be 106 pounds per hour by the tracer technique, was in good agreement with the weighing method.

Further studies by Crickmore and Lean have dealt with the development of a method of measuring sand transport in rivers by means of radioactive tracers. These investigations were also made in a large laboratory flume under equilibrium flow. The transport rate was obtained from the mass distribution of tracer particles initially placed at a depth sufficient to cover all levels of movement of the bed dunes. An attempt was made to explain the shape of the activity distribution curves in terms of a model of the mechanism of transport. The model assumed that every particle has an equal chance of moving a certain hop length. They concluded that a more complicated model which considers both the statistical variation of the hop length and the time that particles spend at different levels in the ripples appears to be necessary.

Sorption of Radioactivity on the Surface of Natural Sediments

Labeling of natural sediments by sorption of radioactivity on the surface has been conducted by a number of investigators. Gilbert and

others (*5*) used sand grains labeled by adsorption (sic) of Ag-110 to investigate sand movement in the Figueira da Foy harbor (Portugal). Silver-110 added to the sand as a solution of silver-110 nitrate was subsequently reduced to metallic silver by sunlight. Under these conditions, 50 to 70 percent of the activity was fixed permanently on the grains. Eight hundred millicuries of this isotope, contained in 4,000 kilograms of sand, was placed on the harbor floor. Rapid dispersion of the tracer rendered a direct survey of the injected area to be of little value. Consequently, samples were collected, the silver was extracted from the sand and analyzed in the laboratory. The long half-life (270 days), together with the added sensitivity gained by sampling and chemical extraction of the isotope, permitted measurements to be made for several months.

No information regarding the mineral composition or the particle size distribution of the sand was included in the assay. The various density and size fractions will show differential sorption of activity. In addition, the sorptive properties of minerals vary considerably. Sand grains behave as individual entities; thus the movement of preferred density or size fractions might well result from such labeling. The conclusions were qualitative in nature. No attempt was made to estimate amounts of transported sediment.

The sorption of gold-198 on natural sediments in the San Francisco Bay was utilized by Krone and others (*15*, *16*, *17*, and *18*) to investigate silt and clay transport in the area. The program of research was concerned with the method of labeling of Bay sediments, detection and measurement of Bay sediments, and the application of these measurements to estuarial sediment transport problems. Three annual progress reports (*15*, *16*, and *17*), and a manual, "An Underwater Scintillation Detector for Gamma Emitters" (*14*), were prepared in conjunction with these investigations. In addition, a final report (*18*) was prepared. The reader is referred to these reports for a detailed and comprehensive discussion of the use of radioactive tracers. Obviously the discussion is primarily concerned with the use of tracers in estuaries; however, it includes a wealth of basic information that is applicable to any sediment tracer program.

The importance of adequate numbers of tracer particles with suitable specific activity was stressed by Krone. Not only is it necessary to supply sufficient activity to overcome background radiation but an adequate number of tracer particles must be seen by the detector. Knowledge of the detector sensitivity becomes increasingly important when quantitative data are desired.

In these reports considerable space was devoted to the selection, calibration, and manipulation of an underwater detector to give quantitative data regarding sediment transport. The in situ measurement of radioactive sediment was performed by continuous monitoring of the estuary floor for tracer abundance. A scintillation detection system was selected, because it would provide the most sensitive and useful response to the tracer. The detector positioned on a sled with lightly loaded runners was found to withstand the vibration, shock, and pressures imposed by the environmental conditions. It was pointed out that the sled was found to be stable and to position the detector consistently with respect to the sediment surface. Vertical adjustment of the detector was possible, permitting detector response to be established for different geometrical locations of the detector to the Bay floor. The sled contained suitable towing harness that was connected to the tow boat, permitting continuous monitoring of the tracer for paths along the estuary floor.

It was determined that the newly deposited tracer was usually confined to a thin layer at the surface of the Bay floor. The detector sled would penetrate any unconsolidated surface material. Once the sled encountered a consolidated surface, it could be supported under the prescribed towing conditions. After considerable inspection and evaluation of the Bay floor, it was proved desirable without exception to position the detector end even with the sled runners for activity measurements in the Bay area.

Detector response is obviously sensitive to the distribution of the tracer material around it. Radiation is attenuated by both water and sediment. These factors make the positioning of the detector, and particularly the consistency of its position with respect to the tracer, the most important factor in any tracer investigation. Maintaining a consistent position of the detector to the Bay sediment placed demanding requirements on consistent towing conditions. If the detector was lifted from the bottom by excessive towing speed, a sharp decline in activity was immediately obvious.

A detector sensitivity of 6,000 counts per minutes per microcurie per square foot was determined in the laboratory for the operating conditions to be used in the Bay measurements. This value was obtained for gold-198 with the discrimination circuitry accepting energies between 0.31 and 0.45 mev, and with the detector positioned with its lower end even with the sled runners.

Towing the underwater detector along selected paths in the labeled area of the Bay permitted activity measurements of the Bay floor. Activity profiles were obtained for the various traverses after correction for the background (measured before addition of the tracer) and decay of the radioisotope. Contours of activity were subsequently drawn. The area between adjacent contours was measured with a planimeter; this area multiplied by the average activity levels between adjacent contours provided activity (in counts per minute) per square foot. Summation of such data divided by the previously determined detector sensitivity (in counts per minute per microcurie per square foot) gave the total number of microcuries deposited. In this manner it was possible to make quantitative estimates of the tracer movement.

The silt transport investigations provided information on the cause of shoaling and the direction of transport. Knowledge of the movements of sediment permitted location of the sources of shoaling material, assisted in evaluating dredging practices and provided guidance in determining remedial measures for the San Francisco Bay Area.

Surface labeling has been used with other radioisotopes in a variety of tracer experiments. The sorption of chromium-51 on sand, $t\frac{1}{2} = 28$ days (0.32 Mev), was used by Davidson (*4*) to study the sand movements in Sweden. It was concluded that this isotope was not satisfactory because of its low gamma emissions and poor penetrating ability.

BaLa-140 (barium-140, $t\frac{1}{2} = 12.8$ days–lanthanum-140, $t\frac{1}{2} = 40$ hours) and lanthanum-140 have been used by Kidson and coworkers (*11, 13*) to label natural flint material for drift experiments at Oxfordness, England. The activity was sorbed on the surface of the pebbles. Lanthanum-140 was particularly suitable for short-lived experiments over part of a tidal cycle. The article discussed the ease of handling these materials and the fact that individual contacts of such pebbles can be recorded. It was also emphasized that material resistant to surface abrasion is required.

Scandium-46, $t\frac{1}{2} = 85$ days (0.89 and 1.12 Mev), has been bonded to the surface of quartz grains by McHenry and McDowell (*19*). A selected size fraction of sand was separated by sieving; the heavy mineral fraction was removed by bromoform and the iron oxide coating subsequently removed by Jeffries' treatment (*10*). Sand prepared in this manner was found to readily sorb scandium-46 from a solution of scandium-46 chloride. Slow heating with infrared light followed by heating to 800° to 1000° C. was sufficient to bond the isotope to the quartz. No measurable change in specific gravity of the quartz was noted for the "tagged" sand. Considerable variation in the amount of activity sorbed by different sand grains was observed when single grains were counted, with the use of a differential scintillation spectrometer. Furthermore, when subjected to leaching and abrasion tests, the removal of activity varied greatly from particle to particle.

Artificial Sediments

Several tracer studies have been performed by the use of artificial sediments. This process involves the incorporation of the desired radioisotope, usually a gamma emitter, in some carrier material, e.g., glass.

Glass containing the suitable isotope has been prepared, ground, and sieved to match the density and grain size distribution of the natural sediment. Activation of the glass by slow neutron irradiation (η, γ) produces a simulated sediment for tracer application.

Radioactive glasses have also been prepared by incorporating the radioisotope with the normal ingredients of glass, melting of these constituents, quenching, grinding, and sieving to obtain the desired size distribution. This technique offers considerable hazard to the personnel preparing the material.

The use of radioactive glass in tracer studies has been particularly informative. A discussion of the studies in part is deemed essential in terms of their application to our sediment problems.

Best known of the various experiments is the pioneer work conducted in 1954–55 on tracing the silt movement in the Thames estuary (*21*). This investigation was conducted to determine if landward transport of silt occurs in the estuary. Some confusion exists regarding the size of the natural sediment in the estuary. Apparently the bulk of the material consisted of aggregates of clay minerals together with appreciable amounts of organic matter. The sizes of the stable aggregates (assuming their stability in the natural environment) are of significance in the preparation and use of simulated sediment particles.

The tracer used in these investigations consisted of soda glass ground and sieved to the grain size distribution obtained by sediment tests on the natural sediment. The median size was 45 microns. The labeled glass was mixed with mud before introduction to the channel bottom. Scandium-46 was the selected isotope because of its energetic gamma emissions, ease of activation in a slow neutron flux, suitable half life (85 days), and the solubility of its oxide in glass.

The large-scale experiment on sediment movement was conducted in 1955. In this test 845 grams of scandium glass ground to a me-

dian size of 45 microns (10^{10} particles) were activated to a level of 29 curies. The material was mixed with mud prior to deposition in the estuary.

The tracer was detected by using three large Geiger tubes designed for underwater surveys. Tracer material moved over distances of 15 miles. Mud in suspension near the bed is deposited and consolidated in the shoal areas. Perhaps the most rewarding information from these studies was noted in the tidal basin at Tilbury Docks (12 miles above the injection point). Siltation in this basin necessitates considerable dredging. Activity levels gradually increased in this area to three times background readings during the first 2 weeks after placement of the tracer. These observations indicated that silt had moved toward the head of the estuary as a consequence of the net landward movement of water close to the bed. Density differences in the river and sea water accounted for the net landward movement of water close to the estuary floor. They concluded that dredged material should not be put back into another part of the estuary but that it should be pumped ashore.

Putnam and Smith (*20*) used scandium-46 glass (prepared in the manner discussed above) to study the movement of sand on the ocean floor. These tests were conducted 2,000 feet offshore in shallow water. Four months after placement of the tracer, after a period of heavy wave action, the tracer was found distributed over an area approximately 3.2×10^5 square feet. Two curies of tracer contained in 2×10^7 particles were used in this study. No quantitative information was given on the rate of transport.

The transport of large "pebbles" (approximately 2 inches in diameter) was investigated by Steers and Smith (*22*). Sandstone pebbles and additional concrete pebbles were obtained to simulate 1,200 naturally occurring pebbles. Holes were drilled in the pebbles and the tracer was added to the interior. The holes were then filled with cement. Barium-140–lanthanum-140 (Ba La-140) was utilized in this study. Monitoring of the pebbles on the sea floor was accomplished by a Geiger-Mueller (GM) detector. Pebbles were located singly by covering the area carefully. This study not only shows that large pebbles are moved on the sea floor but clearly demonstates that individual labeling of gravel or pebbles is feasible. This technique might be applied successfully to the transport of gravel and pebbles larger than 6 or 8 mm. in diameter.

Several aspects of sediment transport by tracer techniques are reported in an extensive article by Arlman and others (*1*). The authors stressed the importance of sufficient number of tracer particles and particle activity to give statistically meaningful results under the conditions of the test, i.e., for the detector sensitivity and the area over which the tracer is dispersed, both horizontally and vertically.

The primary objective was to investigate the movement of sands in the North Sea. Preliminary investigations of sediment transport were conducted in a laboratory flume. Glass beads containing sodium-24, $t\frac{1}{2} = 15.06$ hours (1.37 and 2.75 Mev), were used in this study. Sand was removed from the bed of the flume, mixed with the tracer, and returned to give the same bed configuration. It was desired to operate the flume under equilibrium flow conditions with water discharge, sediment discharge, and sediment input (including tracer) carefully monitored. After the test was started, however, it appeared that the input sediment was not carried off sufficiently from the deposit area. This was apparently the result of a dense packing of the original mixing of the tracer and sand. The equilibrium of the bottom was disturbed, and it was decided to continue the test without replenishment of the deposit area and with decreasing transport of the bed material.

Activity measurements were made from a fixed position on a measuring bridge by the use of a scintillation detector. This detector was shielded with lead to restrict the "field of view" to 300 cm.^{2} on the sand bed. In its fixed position the detector received only photons from a 30-cm. strip of the bed as longitudinal traverses were made. The distribution of activity over other parts of the bed were made on unstirred samples from the bed. Core samples were also removed from the bed and analyzed in a GM counter to obtain information on the vertical distribution of the tracer. This GM counter was found to be inadequate for tracer measurements. Sample preparation was involved. Geometry and back scatter problems made this detector less desirable.

The transport velocities of the glass tracer beads (specific gravity 2.95) and original sand were estimated to be 0.09 and 0.16 meter per second, respectively. The glass beads used in this test did not match the grain size distribution of the sand. The authors concluded, however, that sediment transport could be measured qualitatively and quantitatively by means of radioactive tracers.

According to Kidson and Carr (*12*), measurement of the sand movement in the North Sea was conducted by Arlman and coworkers using scandium-46. It was found impossible to bond the isotope directly to the sand surface; instead, the isotope was bonded to a zeolitic mineral having a specific gravity of 2.72 and 2.76. The zeolite is soft and easily abraided;

grain size would have been reduced from that typical of the natural sand. This was apparently remedied, however, by firing the zeolite to harden it.

Littoral drift measurement of sand with a glass tracer labeled with zinc-65, $t\frac{1}{2} = 245$ days (1.12 Mev), was made in Japan by Inose and others (*9*). Zinc-65 was chosen because of its (1) strong gamma emissions, (2) long half-life, (3) low cost, and (4) ease of incorporation in glass. The isotope had to be shipped from the United States and be incorporated in glass before use; thus, the 245-day half-life was not undesirably long. No irradiation facilities were available in Japan, making it necessary to fix the radioisotope in the glass directly.

The radioactive melt, carefully prepared to give the proper specific gravity 2.63, was quenched in water, crushed, and sieved to obtain the desired size distribution. The addition of the radioisotope before grinding is not desirable unless special facilities are available for protection of the personnel.

Thirty millicuries in 1.5 liters of ground glass (approximately 10^9 to 10^{10} particles) were used in the investigation. Measurements were made with a scintillometer using a 1 by 1 by 0.5 inch anthracene crystal. The tracer was placed in 2.8 meters of water. Four radial surveys from the injection point were made during the month after deposition of the tracer. Activity measurements were plotted as maps of equal activity rate contours. The direction of net transport was readily obtained from contours plotted for the different surveys.

Measurement of the Tracer

Interaction of Radiation With Matter

Sediment transport investigations using labeled sediment requires appropriate instrumentation capable of detecting and measuring the intensity of the radiation. Isotopes emitting only alpha or beta particles would not be suitable for in situ measurements of sediment transport. These particles are readily absorbed in matter. For illustration, examples of the penetrating abilities of hard and soft beta radiations are given in table 1.

TABLE 1.—*Penetrating abilities of hard and soft beta radiations*

Isotope	Maximum energy	Penetrating ability		Radiation range
		In air	In water	
	Mev	*Inches*	*Inches*	*Mg./cm.²*
Phosphorus-32	1.70	280	0.2	800
Carbon-14	.155	10	.01	28
Yittrium-90	2.18	350	.45	1,100

These data indicate the impracticability of using even the hard betas for in situ measurements of sediment transport. Phosphorus-32, $t\frac{1}{2} = 14$ days (1.7 Mev), or yittrium-90, $t\frac{1}{2} = 64.2$ hours (2.18 Mev), could be used in the manner employed by Inman and Chamberlain. The use of gamma-emitting isotopes, which has been stressed by previous workers, is necessary where measurements are to be conducted with a minimum of sampling.

When gamma radiation interacts with matter, there are essentially three competing possibilities; namely, the photoelectric effect, Compton interaction, and pair production. These processes are dependent upon the energy of the radiation and the nature of the absorber. It will suffice to state that in the photoelectric process essentially all the energy of the gamma photon is transferred to an electron that is ejected. This is a true absorption effect. Likewise, in pair production the gamma ray energy goes into the formation of a pair of electrons and imparts kinetic energy to them. In the Compton interaction, however, only a part of the energy of the incident photon is transferred to the electron, since a scattered photon of lower energy is also produced. This lower energy photon may suffer several Compton collisions, i.e., undergo multiple scattering, depending on the thickness of the absorber. If it does not escape, the scattered photon will eventually be absorbed as a result of the photoelectric effect.

In passing through matter, gamma ray photons are absorbed so that the intensity falls off in an exponential manner. If a collimated beam of monoenergetic gamma rays of intensity I_0 passes through a thickness $\times$ absorber, the intensity I_x of the emergent beam is given by the familiar equation.

$$I_x = I_0 e^{-\mu x} \qquad (1)$$

where μ is the linear absorption coefficient of the absorber for the given radiation. It should be noted that the absorption coefficient is a measure of the probability of occurrence of an interaction process. The total absorption coefficient μ_0 is the arithmetic sum of the various components of interaction discussed above. Laboratory calculations frequently are made on the thickness of the material that reduces the radiation intensity by one-half. This "half thickness" or "half value thickness" ($X_{\frac{1}{2}}$) is related to the absorption coefficient by

$$X_{\frac{1}{2}} = \frac{0.693}{\mu_0} \qquad (2)$$

These values are valid only for absorption obeying a simple exponential function, i.e., for a narrow collimated (parallel) beam of monoenergetic radiation. For this so-called "good" geometry, it is postulated that scattered radiation is completely removed from the radiation

beam. In other words, the scattered radiation does not reach the detector.

Absorption and geometric attenuation of radiation becomes important in tracer applications where the activity measurements are usually performed under "broad beam" or "poor" geometry conditions. In this situation, the scattered photons are not removed and the intensity at the detector is greater than that estimated by the exponential equation. The difference between the results of measurements in "narrow" and "broad beam" cases is described by means of a buildup factor. This factor refers to some measurable property of the radiation flux (e.g., intensity) and is defined as the ratio of this quantity when the effects of all photons are included to that obtained when only the uncollided photons are considered. Although the interaction of gamma radiation with matter is well defined, the calculation of buildup factors depending on the multiple scattered radiation is still a difficult problem. The correction of measured values for buildup can, nevertheless, be made in many cases.

Detector System

Both Geiger-Mueller (GM) and scintillation (organic and inorganic) detectors have been utilized for activity measurements of the labeled sediment. A complete treatise on the various detection systems available is beyond the scope of this paper. However, a brief discussion of GM and scintillation detectors, relevant to their application to sediment tracing, is deemed necessary.

Geiger-Mueller detectors are ion collection devices (consisting usually of an enclosed metal cylinder with a fine center or concentric wire electrode) containing an ionizing and quenching gas. Ionization is produced when radiation passes through the gas-filled chamber. A high potential is placed across the electrodes to allow collection, acceleration, and subsequent production of secondary ions on collision with the gas atoms. An avalanche of ions results that produces a current through the electrode system. The voltage across the electrodes is sufficient to produce gas amplification in the detector of 10^7. Under these conditions the discharge covers the entire length of the central electrode and all the output pulses are of the same size or height. The height of the pulses produced are consequently not proportional to the energy of the incident radiation. These pulses are amplified and counted or integrated with time, to measure the intensity of the incident radiation.

The efficiency of the GM detectors for gamma photons is 0.5 to 2 percent. High energy photons often penetrate the ion collection chamber without producing ionization. Furthermore, these detectors are characterized by a long resolving time, i.e., the minimum time interval required between registered events. Typical resolving times for GM detectors are in the range of 200 to 600 microseconds. Such systems have considerable loss of counts when receiving events greater than 10,000 counts per minute. Corrections can be made, however, to the registered events once the resolving time is determined for the counting system.

GM detectors are simple, rugged, and can be produced in large sizes. They require only a stable power supply and rate meter for field use. The disadvantages of this detector system are, however, the inability to distinguish or discriminate between photon energies, low efficiency for gamma radiation, and limited useful radiation intensity range.

Scintillation detectors utilize a principle different from the ionization-collection technique employed in the GM detector. Gamma radiation transfers energy to the electrons of matter with which it interacts. This energy transfer may be total or partial, depending on the type of interaction. The excited electrons in turn lose energy either by ionization or excitement of the atoms through which they are passing. This excitation of the atoms can be used for detection of the incident gamma radiation. Substances, i.e., phosphors, are available that can absorb such excitation energy and reemit (fluoresce) the energy in the form of light photons. A sodium iodide (thallium activated) crystal is the most commonly used phosphor for detecting gamma radiation. The important feature of this type of detection is that the number of light photons produced is proportional to the energy of the electron (i.e., to the absorbed energy). Light photons produced in the phosphor are converted to electrical energy by a multiplier phototube. These pulses undergo considerable amplification and are finally recorded by suitable scaling equipment.

Incorporation of a pulse height analyzer in this system permits rejection of unwanted emission energies. Ideally, two discriminators — contained in the pulse height analyzer — can be employed to accept only those pulses corresponding to the characteristic gamma photon energies of the particular radioisotope. This system permits (1) the rejection of already existing activity, i.e., background, having emission energies different from the tracer, (2) rejection of scattered radiation from attenuation of the photons on passing through matter, (3) use of different tracers simultaneously, i.e., radioisotopes having well-defined energy spectrums whose primary photon energies can be separated, (4) the accurate measurement of high levels of activity without loss of counts

from "dead" time of the system, and (5), within limits, the calibration of the detector with regard to spatial distribution of the tracer.

A complete scintillation counting system is made up of the following components:

(1) Detector (scintillation probe) — sodium iodide (thallium activated) crystal, multiplier phototube, and preamplifier;
(2) Pulse amplifier;
(3) Pulse height analyzer — upper and lower discriminators;
(4) High voltage supply;
(5) Scaler and timer; or;
(6) Ratemeter;
(7) Strip chart recorder.

These systems require linear pulse amplification and very stable voltage supplies. They are obviously lacking in ruggedness and require careful handling during field surveys. They do offer the ultimate in instrumentation for gamma ray detection in any tracer program.

The sole purpose of instrumentation is to determine the amount of unknown radioactivity for a particular system. To make full use of the scintillation system, it is possible to calibrate the instrument to determine the counting rate under the photopeak energy of a given radioisotope. This can be illustrated by referring to the gamma ray spectrum of scandium-46 in figure 1. The two photopeaks, 0.88 and 1.12 Mev, can be used to advantage in activity surveys. With the appropriate analyzer settings, e.g., 1.02–1.22 (2-volt gate) Mev for scandium-46, the counting rate would be essentially independent of scattered radiation in the source or in the vicinity of the crystal.

Measurements of the activity can then be conducted by one or a combination of the following procedures: (1) The absolute measurement of gamma ray activity by determining the probability of a maximum interaction in the given size of crystal and calculating the efficiency of detection for that energy, (2) a relative measurement may be made by counting in the photopeak region and calibrating the instrument with a sample of known activity intensity, and/or (3) under certain experimental conditions a relative measurement may be made by counting in the photopeak for the unknown and reference samples. In the latter case the actual activity of the reference sample need not be known, the interest is placed on the change in the reference before and after (or during) the experimentation. In any situation, constant attention must be given to maintaining the same source geometry. Ideally, it is desirable to apply all these methods of counting standardization so that one may serve as a check on the other. In other words, it is advisable to use a standard source of desired radioisotope and determine the resolution, sensitivity, and efficiency for the counting system under differing geometrical arrangements of detector to source.

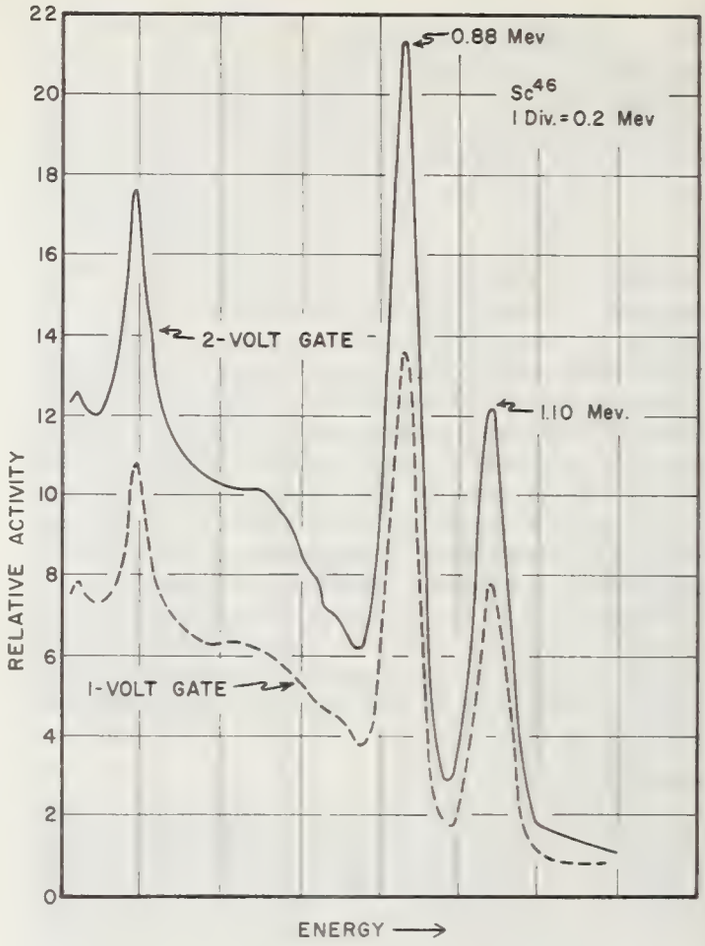


FIGURE 1. — The gamma ray energy spectrum of scandium-46. (Differential scintillation spectrometer: 1- and 2-volt gates.)

Sediment Transport in a Laboratory Flume

Radioactive tracers are being used by the Sedimentation Laboratory to investigate sediment transport by both overland and channel flow conditions. Controlled investigations utilizing tracers will be conducted in a 200-foot outdoor flume. These studies will provide needed information for the design, analysis, and interpretation of current and future field investigations.

Before initiating studies in the 200-foot flume, it was decided to conduct a preliminary investigation of sediment transport in one of the small flumes located in the Hydraulics Laboratory. A differential scintillation spectrometer has recently been modified for flume and field investigations of tracer movement. A preliminary study of sediment transport in a small flume under low flow conditions would (1) provide a check on the operation of the detecting and

counting system, i.e, the spectrometer system; (2) provide information on the procedures to use in determining tracer distributions, both vertically and horizontally; (3) acquaint the personnel with the problems associated with the geometry of the detector and source; and (4) illustrate the overall difficulties associated with a quantitative analysis of tracer distribution.

Methods and Materials

The flume selected for this study consisted of a rectangular channel 50 feet long and 14 inches wide equipped with a variable speed axial flow pump at the downstream end and a return pipe for recirculating water and sediment to the upper end of the channel. The flume was temporarily modified to permit operation as a "put and catch" channel, i.e., water could be added at the upper end and water and sediment discharges could be measured at the downstream end.

An instrument rack mounted on rails along the upper edge of the flume provided a mount for point gage and tracer detection equipment. The carrier with point gage provided a reference for positioning the detector. Longitudinal traverses of the flume could be made by moving the instrument rack along the rails. Thus, the detector, i.e., a scintillation probe, could be positioned vertically or horizontally at any point above the channel bed.

The scintillation probe (3 inches in diameter and 14 inches in length) contained a 2- by 2-inch sodium iodide (thallium activated) crystal, multiplier phototube, and preamplifier. The probe was connected through 100 feet of cable to the differential scintillation spectrometer. The cable was necessary to (1) supply voltage to the multiplier phototube, (2) supply voltage to the preamplifier, and (3) receive the incoming pulses from the preamplifier. The discriminators (contained in the pulse height analyzer) were calibrated to accept only pulses corresponding to photons having an energy range of 1.02–1.22 Mev. This energy corresponds to the second photopeak in the scandium-46 energy spectrum. The pulses, after amplification and discrimination, were supplied to a ratemeter for integration with time or to a scaler-timer unit. Provision was also made to supply the ratemeter output to a strip chart recorder for permanent recording of the tracer signal.

The upper 10 feet of the channel were used to dissipate the energy of the incoming water and hence to provide a smooth flow condition to the channel bed. The flume was operated at zero slope. Sand having a mean diameter of 0.48 mm. was used as the bed material. The sand bed was smoothed under conditions of low flow to give a slope of 0.0025. Water depths ranged from 0.017 to 0.035 foot.

Ten grams (10^5 grains) of radioactive sand (0.350 to 0.297 mm. in diameter) were prepared by the following method:

1. Heavy minerals (specific gravity, 2.89) were removed by separation in bromoform.
2. Iron oxides were removed from the surface of the quartz grains by Jeffries' treatment, i.e., reduction with magnesium in oxalic acid.
3. Further separations in bromoform were made to obtain sand having a specific gravity of 2.60 to 2.65.
4. The sand was activated by the sorption technique. The sand was permitted to sorb scandium-46 chloride with subsequent heating to 840° C. to convert the chloride to the more stable oxide.
5. The activated sand was washed in water to remove any loosely bonded radioisotope.

Previous studies have indicated that considerable variation in grain activity is obtained with this system of labeling. Furthermore, it has been shown that the loss of activity by abrasion in water and sand varies from one grain to another. Estimates of activity lost under laboratory abrasion tests range from 2 to 5 percent. Obviously it is difficult to estimate the losses that may occur under field or flume conditions.

The labeled sand was mixed with 40 grams of nonradioactive sand and subsequently placed in a 2- by 14-inch pan for analysis. The detector, placed at a distance of 0.12 foot above the sand, was used to obtain measurements of the radiation. Attenuation measurements of the radiation were also made with varying depths of water above the sample.

The labeled sand was subsequently placed on the surface of the channel bed 15 feet from the upstream end of the flume. Placement was made in a narrow band extending the width of the flume. Activity measurements of the injection strip were in agreement with the measurements conducted in the analysis pan. Measurements made on the upstream part of the channel and 33 feet downstream from the injection strip indicated a background radiation of two counts per second for the specific spectrometer operation.

Water discharge measurements were periodically checked and found to vary from 0.0146 to 0.0150 cubic foot per second during the 5 days of operation. Sediment transport was extremely slow during the experiment. The sediment discharge for the first 30 hours of flume operation was 7 grams per minute. Trapping problems arose during the second day; measurements of the sediment discharge were terminated for the remainder of the experiment. During the experiment, considerable change in the bed configuration occurred. Small sand dunes,

¼-inch in height and 2 to 4 feet in length, were present during the second day of flow. This condition slowly changed to a meandering thalweg that remained throughout the experiment. The thalweg extended along one flume wall from station 0 to station 7. It crossed the channel at this point to the other flume wall and continued downstream.

It was originally planned that the probe would be surrounded by a lead shield to restrict the area of the crystal receiving incoming radiation. The area of the channel bed "viewed" by the detector could be well defined by extending the lead shield below the crystal surface. Two inches of lead are required to attenuate 99 percent of the gamma photons having an energy of 1.10 Mev. Shielding of this magnitude presents two immediate problems: (1) The physical manipulation of the probe, and (2) attenuation losses of the radiation in the additional 2 inches of air interposed between the crystal and the channel bed. The latter condition would make it necessary to increase the activity level in the bed. Restriction of the channel bed area "viewed" by the detector is advantageous; it, however, does necessitate the use of larger amounts of activity to obtain sufficient flux for the crystal. It was desired to keep the activity levels to a minimum; hence the lead shield was not used in this trial run.

A ratemeter, having a range in time constants from ½ to 80 seconds, was used in various parts of the study. This instrument contains both a panel-type meter and a recorder jack for reading the output. It has the advantage of indicating and recording the counting rate continuously. It cannot, however, attain the accuracy of the counter-timer (scaler-timer) combination for situations in which the measuring time is limited.

Results and Discussion

Horizontal and Vertical Distribution of Activity

Measurements were made with the probe in a fixed position above the bed and at 1-foot intervals downstream from the injection strip. The ratemeter is, of course, less desirable than the scaler-timer combination for these measurements. Sufficient time must elapse for equilibrium to be established in the tank circuit of the ratemeter before the errors involved are entirely statistical. Activity surveys of the channel bed were made after 5, 23, 30, 46, 70, and 125 hours of flow. The measured activities at 1-foot intervals were related to the total activity placed in the injection strip to obtain the fraction of the labeled sand at each cross section of the channel (figs. 2 and 3). These measurements were made with an unshielded probe at a constant distance from the channel bed. No corrections have been applied to the data for the

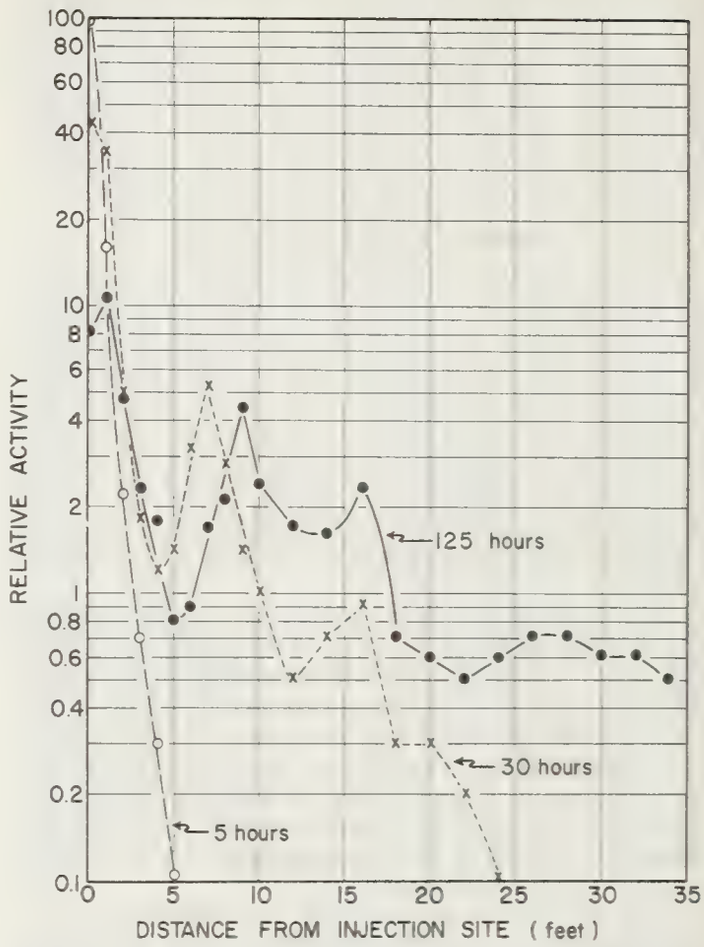


FIGURE 2. — Distribution of scandium-46 labeled sand in the channel bed after 5, 30, and 125 hours of flow.

statistical errors of the independent measurements. Such considerations will be necessary for quantitative analyses in which the summation of a series of independent measurements are compared to the originally deposited tracer.

Summations of the activity surveys for 5 and 23 hours show a positive error of roughly 17 percent. This is to be expected during the early stages of the experiment when the activity gradient is steep. A rough approximation can be applied to determine the contributions to the detector from adjacent areas. Such approximations indicate that an error of roughly 10 percent exists between the original measurement of the deposited tracer and the summed values of the individual measurements.

The data in figures 2 and 3 represent a semiquantitative analysis of the tracer distributions for various flow time intervals. If quantitative data were available it would be possible to estimate the transport velocity of the labeled sand from the weighted distributions of tracer for any given activity survey. The calculation of sediment-transport velocity and sediment discharge will be discussed further in a later part of this section.

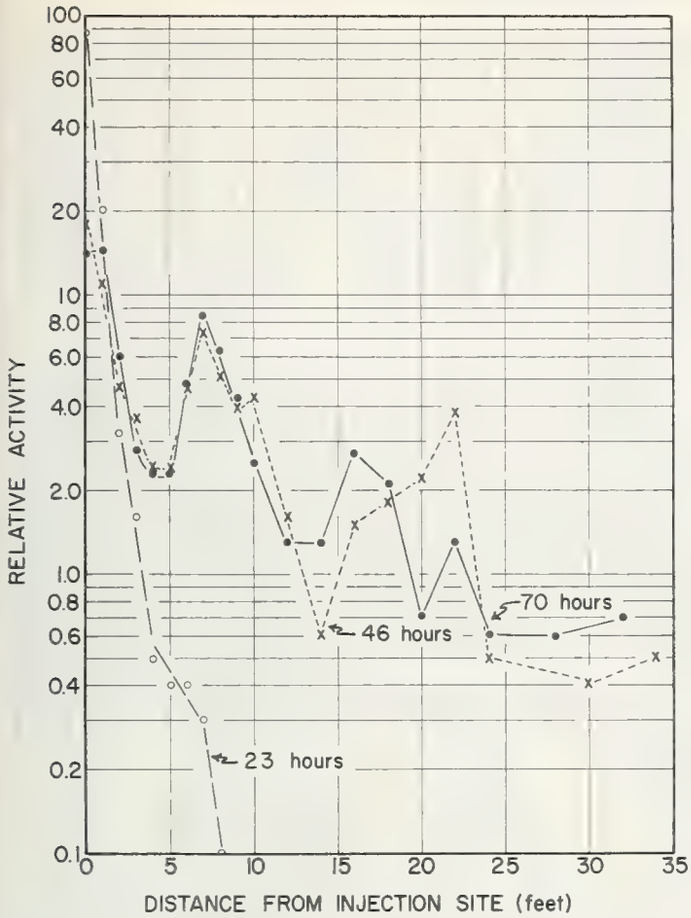


FIGURE 3. — Distribution of scandium-46 labeled sand in the channel bed after 23, 46, and 70 hours of flow.

Sediment transport velocities can be obtained by monitoring the injection strip for equilibrium flow conditions. In this experiment where aggradation and degradation are occurring, such measurements are questionable. Decreases in activity in the injection strip, as noted by the detector, may be the result of (1) the actual movement of tracer from the area, and/or (2) the attenuation of gamma photons from the tracer by overlying sediment. The detector is unable to distinguish between a few photons resulting from a large quantity of buried tracer and those arising from a small amount of tracer on the surface of the bed. The vertical distribution of tracer is then one of the more important factors in any tracer analysis. Qualitatively, there is evidence of tracer burial by nonradioactive sand in the 30- and 46-hour activity surveys. Observations of the changing bed configuration during the flow also lead to this conclusion.

Considerable variation of activity within each cross section was observed during the flow. These measurements were made by traversing definite cross sections of the channel. A more detailed survey of the variation of activity was made after termination of the flow. Variations within the injection strip and the first five stations downstream are illustrated graphically in figure 4. The values show qualitatively (1) the presence of high activity levels in the upstream end of the channel, and (2) variations of three- to fourfold for activities within any one cross section. The values represent the total radiation as received by the detector without any correction for attenuation of photons from buried tracer. The lower activities present in the 8.75- to 12.25-inch part of the channel bed reflect the removal of tracer in the thalweg. The higher activity levels at station 7 (fig. 2) illustrate the cross-over point of the thalweg. The cross-over point of the thalweg progressed downstream at the rate of 0.02 ft./hr. as determined from the 30- and 125-hour activity surveys.

The analysis of any tracer distribution is necessarily complicated by nonuniform burial of the tracer. The vertical distribution of tracer was qualitatively determined at various cross sectional intervals. This was accomplished by removing thin layers of sand from the bed with subsequent measurements of the activity levels

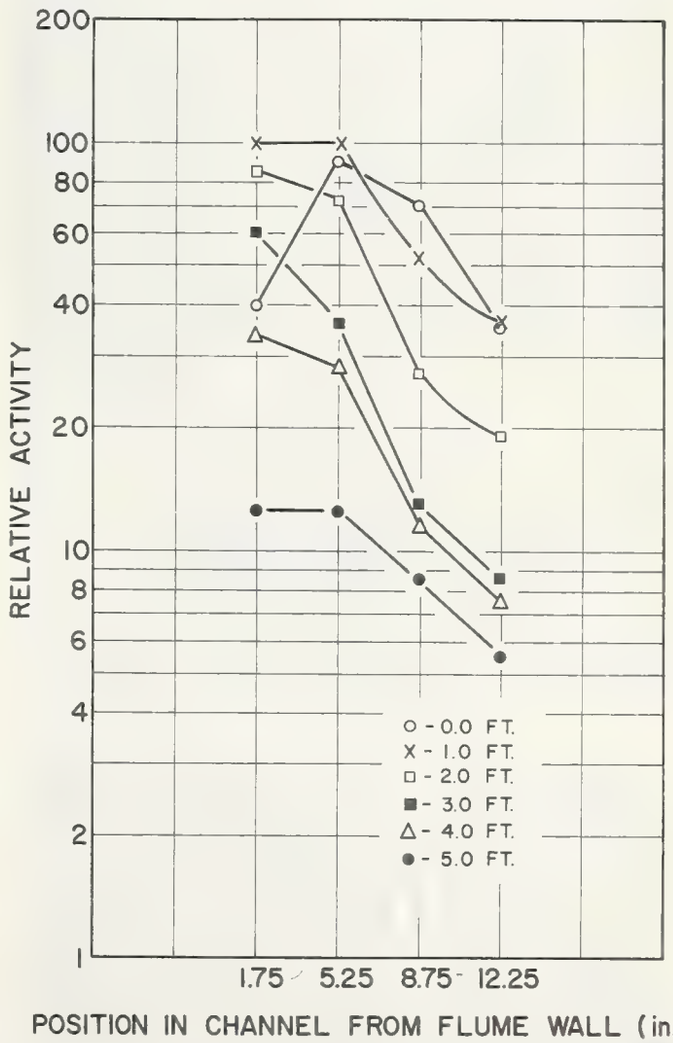


FIGURE 4. — Distribution of scandium-46 labeled sand within defined sections of the channel bed.

remaining in the bed. The depth of sediment removed each time was determined by point gage measurements of the channel bed. Surveys of stations 0, 1, and 2 are given in figures 5, 6, and 7, respectively. Only two positions within

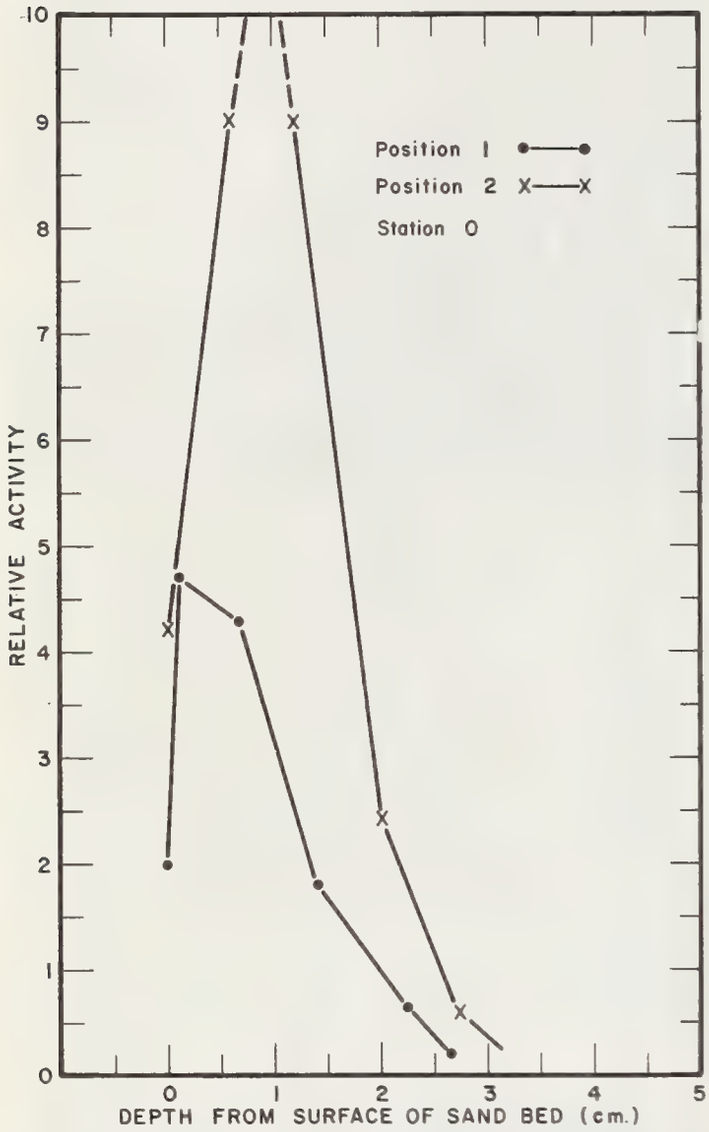


FIGURE 5. — Vertical distribution of scandium-46 labeled sand in the channel bed after 125 hours of flume operation (station 0).

each cross section are graphically illustrated. The activity levels at zero depth refer to those determined on the natural channel bed prior to the removal of sand. The initial increase in activity (as seen by the detector), when successive vertical layers of sand were removed, is indicative of aggradation in this part of the channel. The photons from the tracer were attenuated by the overlying nonradioactive sand. Tracer sand was covered by approximately 1 cm. of sand for this part of the bed.

The decrease in activity with successive removal of bed material is to be expected. These data clearly indicate that tracer sand was buried to a depth of 1.0 to 1.2 inches for the flow condition imposed on the bed. Vertical examination of stations 3, 4, 5, and 6 indicated no overlying nonradioactive sand, but values for buried tracer were similar to those mentioned above. Further examination of the bed at various cross sections indicated that labeled sand had been buried to a depth of 1.0 to 1.2 inches throughout the channel bed.

Measurement and Correction of Attenuation

Vertical distribution of tracer places a serious restriction on the investigation of sediment transport by tracer application. Knowledge of the attenuation of the gamma photons in passing through the sediment is essential for any quantitative estimate of sediment transport. In this experiment the attenuation of the 1.12 Mev photons by sand (bulk density 1.7) from scandium-46 was measured by a plane source (cross

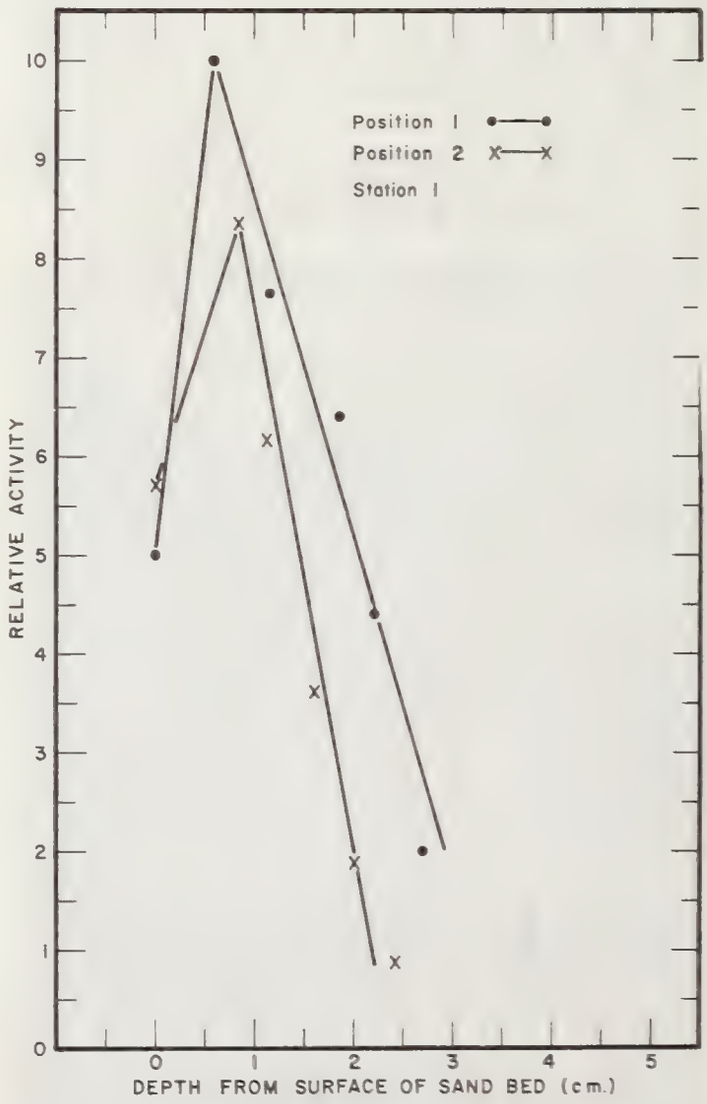


FIGURE 6. — Vertical distribution of scandium-46 labeled sand in the channel bed after 125 hours of flume operation (station 1).

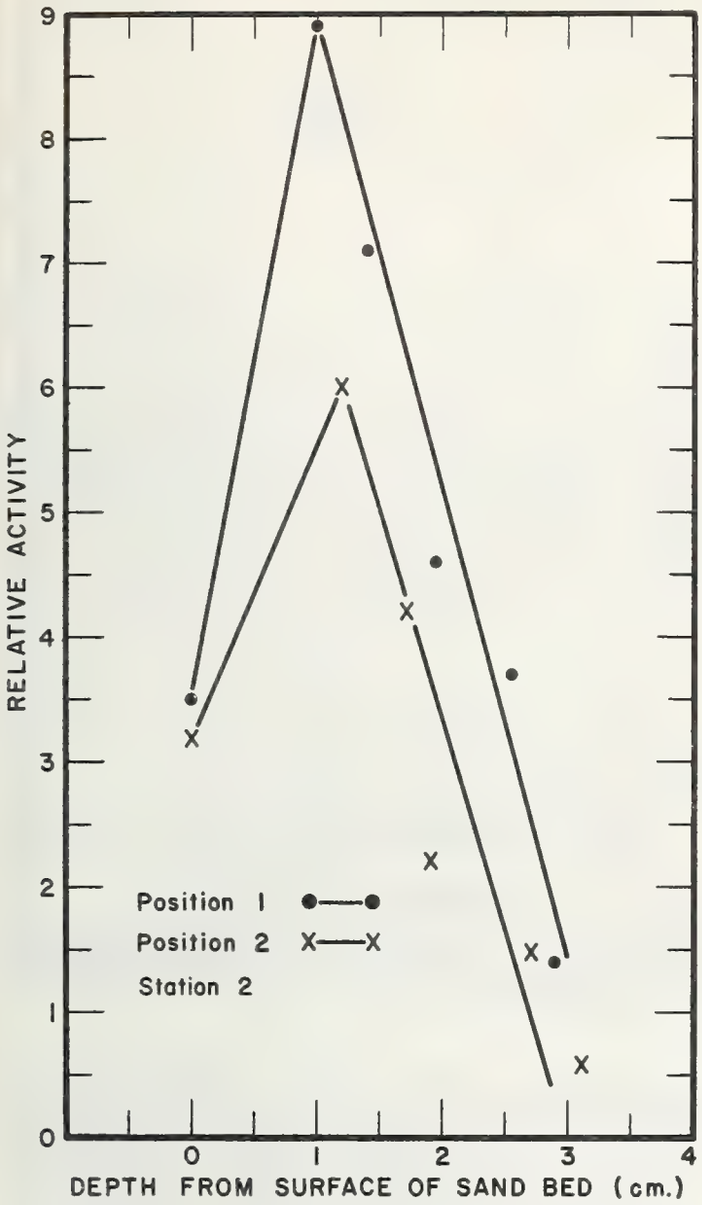


FIGURE 7.—Vertical distribution of scandium-46 labeled sand after 125 hours of flume operation (station 2).

sectional area—8 cm.^{2}) of labeled sand. Measurements were made by interposing various thicknesses of sand between the source and detector. These determinations were made under "poor geometry" conditions. The results are presented in figure 8. The half-thickness value, i.e., the thickness of sand required to attenuate one-half of the incident photons, was found to be 2.5 cm. as measured to the surface of the probe. Extrapolation of this value to the center of the phosphor would give a value of 5 cm. under these particular operating conditions. The tenth-value, i.e., the amount of sand required to attenuate 90 percent of the photons, is found to be 9.5 cm. when extrapolated to the center of the detecting crystal.

The total linear absorption coefficient for sand can be calculated from equation 2:

$$\mu_o X_{1/2} = 0.693$$

$$\mu_o = \frac{0.693}{5} \qquad (3)$$

$$\mu_o = 0.138/\text{cm.}$$

Linear and mass absorption coefficients are related in the following manner:

$$\mu = m\rho \qquad (4)$$

where μ and m refer to the linear and mass coefficients, respectively, and ρ refers to the density of the absorber. The linear (or mass) absorption coefficient of water is 0.065/cm. (or cm.^{2}/gm.) for 1.1 Mev gamma rays. This value can be taken as the mass absorption coefficient of sand. Equation 4 can, therefore, be used to estimate the linear absorption coefficient of sand, bulk density 1.7 g./cm.^{3} The value of $\mu =$ (0.065) (1.7) becomes 0.11/cm. which is in reasonable agreement with the experimentally determined value of 0.14/cm.

Knowledge of the "average thickness" of burial combined with the linear absorption coefficient of the sediment for the particular gamma ray energy would permit a correction to be applied (equation 1) for attenuation losses of buried tracer. Such a correction assumes a uniform vertical distribution of tracer as illustrated in figure 9, *a*. If the tracer is overlain by sediment as shown in figure 9, *b*, a correction for attenuation losses for both the zone of uniform burial and the overlying sediment could be applied. The latter may be calculated from the following relationship as derived by Glasstone (*6*).

$$f = \frac{\lambda}{2H}\left[\frac{1}{2} - F_2\frac{(H)}{\lambda}\right] \qquad (5)$$

where f = fraction of radiation escaping from a self-absorbing slab source;
λ = the relaxation length of the shielding for the given radiation = $1/\mu$;
μ = linear absorption coefficient;
H = thickness of the source;
$F_2(q) \equiv q^2E_3(q)$;
$E_3(q) \equiv \int_q^\infty \frac{\exp(-p)}{p^3}\,dp$;
$q = H/\lambda$;
$p = R/\lambda$;
R = distance between source and detector.

Calculation of Sediment Discharge

Equations 1 and 5 can readily be applied to the activity survey of the channel bed after 125 hours of flow. Roughly, 55 percent of the activity present at stations 0, 1, and 2 was received by the detector. Forty-two percent of the total activity initially placed in the channel (station

0) remained in stations 0, 1, and 2 after 125 hours of flow; the remaining 58 percent of the activity was distributed throughout the channel reach, i.e., 34 feet. An average sediment transport velocity (V_s) of 0.0688 ft./hr. is obtained from the weighted longitudinal distribution of tracer (corrected for vertical distribution) for the 125-hour survey. An estimate of the sediment discharge (Q_s) for the 0.35- to 0.297-mm. sand can be made from the following relationship:

$$Q_s = W\ d\ V_s\ f_s\ a_s \qquad (6)$$

where Q_s=sediment discharge, pounds per hour;

W=width of channel=1.16 ft.;

d=average depth to which sediment movement occurred=0.0917 ft.;

V_s=average sediment transport velocity=0.0688 ft./hr.;

f_s=fraction by weight of 0.35 to 0.297 mm. sand found in the transported sand=0.08;

a_s=bulk density of the sand = 1.06 lb./cu. ft.

A sediment discharge value of 0.062 lb./hr. is obtained by calculation. The measured discharge rate (0.35- to 0.297-mm. fraction) was found to be 0.074 lb./hr. for the first 30 hours of flow.

Future Investigations

The wooden flume (1 by 1 by 200 feet) will be operated on a "put and catch" system for both water and sediment. Early investigations will be restricted to low flow conditions with some analyses of sediment transport attempted during the flow, but with the major emphasis placed on the horizontal and vertical distribution of tracer after termination of flow. Both equilibrium and nonequilibrium flow conditions will be investigated. Emphasis will be placed on the quantitative analysis of the total tracer. The differential scintillation spectrometer described herein with scaler-timer combination will be used for the analyses of the tracer distribution. Several specific conditions and techniques will be considered. They may be briefly outlined in the following manner:

(1) The use of short half-life isotopes will alleviate the problem of storage of contaminated sand. Several possible isotopes are available with a wide range of gamma photon energies. The lower energy gamma photons will be adequate for these investigations, because emphasis will be placed, at least in the beginning, on the analyses of dry sand beds (i.e., after termination of flow).

(2) If available, radioactive glasses will be used as tracers for the bulk sand. Such glasses

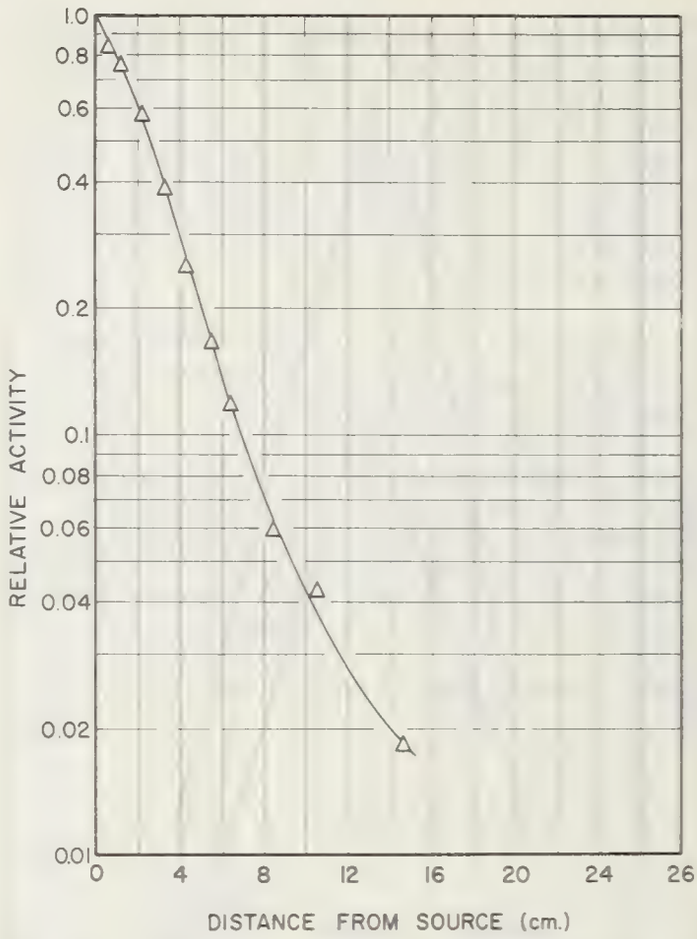


FIGURE 8. — The attenuation of scandium-46 photons by sand (bulk density 1.70 g./cm.^{3}; differential spectrometer; 1.12 Mev peak, 1-volt gate).

are currently available (with a variety of isotopes) in certain restricted density and size ranges. The preparation of special melts (specific gravity 2.65) with the desired isotope has been under investigation for several months. Radioactive glass produced from a single melt (batch) would possess the following desirable properties:

(a) It would be mass labeled, i.e., the specific activity per gram of glass would be constant throughout the batch. The photons received by the detector would be proportional to the mass of sediment transported.

(b) The melt can be crushed, ground, and sieved to produce a desired grain size distribution.

(c) The physical and chemical stability of such glass greatly reduces the possibility of tracer loss by abrasion and dissolution.

(d) A standard melt would provide material for the standardization of the detection system. In other words, glass containing a known specific activity per gram could be obtained for measurement of the sensitivity and efficiency of

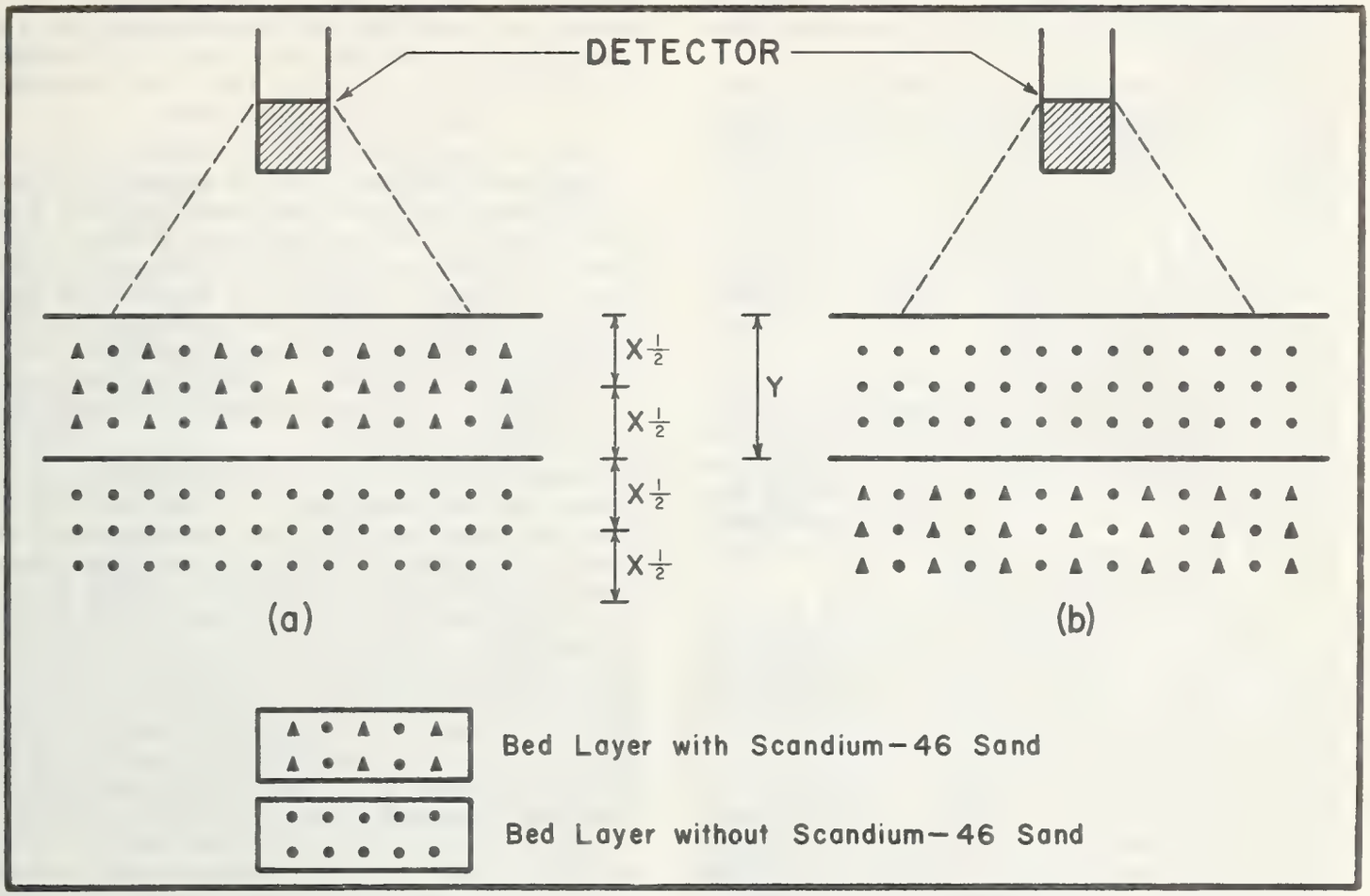


FIGURE 9. — Ideal vertical distribution of labeled sand: *a*, Uniform burial without overlying sediment; *b*, uniform burial overlain by nonradioactive sediment.

the instrument. Such material would be invaluable for investigating various geometries and spatial distributions of tracer.

(3) Lead shielding will be employed to define the geometry of the detector and source, i.e., tracer in the channel bed. It may be necessary to replace the present 2-inch shield with a smaller one which is more easily manipulated by point gage equipment. The thickness of the shield will depend upon the energy of the particular isotope.

(4) Transport studies of selected grain sizes and total size distributions matching the bulk sand will be considered.

(5) Analyses of the channel bed for both horizontal and vertical distribution of tracer will be conducted. Total analyses will be attempted. In other words, a summation of the activity levels from various cross sections of the bed (as seen by the detector from a well-defined geometry above the bed) will be compared to the initial amount of tracer placed in the channel. The attenuation of photons from buried tracer will cause the summed values as detected above to be smaller than the initial amount of deposited tracer. Careful measurements of the vertical distributions will then be conducted by removing thin layers of sand from the bed with subsequent measurement of the activity present in each layer and measuring the activity remaining in the bed.

In some situations it may be possible to correct partially for the attenuation losses of the buried tracer. Corrections will be made, assuming uniform vertical distributions of tracer, by use of equations 1 and 5. Situations involving nonuniform vertical distributions of tracer combined with interstratified layers of nonradioactive sediment that are most likely to be under these conditions are speculative. Nevertheless, it will be informative to make such approximations, using a model distribution, and to compare the corrected values of previously measured activities with the total amount of tracer initially placed in the channel.

(6) In the majority of experiments, the tracer will be mixed uniformly with sand from a definite cross section in the channel bed and replaced to simulate the original bed configuration. The tracer applied in this manner will

appear to the detector as a volume-distributed sample with self absorption, i.e., represented by equation 5.

(7) The hydraulic parameters will be measured so that tracer distributions will be meaningful. Comparisons of the calculated and measured sediment discharges will be informative.

(8) The average transport velocity of the sediment can be calculated from the weighted distribution of the tracer; subsequent calculations of the sediment discharge can then be made by using equation 6.

The ultimate objective of the tracer program is to investigate sediment transport in the field. These measurements in both overland flow, i.e., gross erosion, and in channels will require quantitative analyses. Certainly there is little need for qualitative evaluations of sediment transport in agricultural watersheds, e.g., the directional movement of sediment.

The spectrometer described herein, mounted in a vehicle with auxiliary power supply, will be used for these field investigations. The scintillation probe will be connected through 500 feet of cable to the spectrometer. This will facilitate measurements of activity in areas nonaccessible by the vehicle. Vertical core samples of the particular area, "fixed" in gelatin or in polyester resins to keep them intact, can be returned to the laboratory for further analyses. This will provide some information on the vertical distribution of the activity.

Analysis of the vertical distribution of activity will be necessary not only for estimating the amount of buried tracer but also in defining the average depth of particle movement, d, that is the zone through which sediment transport has occurred. The analysis and interpretation of the "average" or "effective" depth of particle movement and tracer distributions will be extremely complex for nonequilibrium events. The analysis is simplified considerably for an equilibrium flow, e.g., where the bed configuration consists entirely of dunes of a certain mean amplitude, with no net accretion or scour occurring in the bed.

Problems associated with the determination of d can be more clearly illustrated by considering an application of radioactive tracers of specific interest to the Agricultural Research Service, i.e., the measurement of bedload discharge in ephemeral sand bed streams. For example, assume that a definite cross section (injection strip) of the channel bed is uniformly labeled with the tracer. Furthermore, assume that (1) the tracer is placed to a depth sufficient to embrace the deepest dune troughs present during the flow, and (2) the tracer sand is mass labeled, i.e., that the total size distribution of sand found in the channel bed is labeled so that the activity detected at any point downstream after the flow has subsided will be proportional to the total mass of tracer and not to any specific size fraction.

During any flow event, considerable change in bed configuration might be expected. Dunes of different amplitude and duration plus local scour (and fill) with subsequent removal (and burial) of tracer from the injection strip will occur. A certain fraction of the labeled sand will be found distributed downstream from tne injection strip after termination of flow. Vertical core samples will be taken from the injection strip at frequent intervals extending the width of the channel.

Assume that the vertical distribution of the tracer can be quantitatively measured for all the core samples. Each core sample could be fixed in a polyester resin for subsequent analysis in the laboratory under well-defined analytical conditions. These cores can be segmented at various intervals, and the activity present in each segment compared to a like segment in the nondisturbed, uniformly distributed portion of the core sample. The vertical distribution of activity in each core will represent the total or overall series of events that occurred during the flow. Changes in bed configuration will be reflected in the activity distributions of the selected core samples. For example, certain depths will be noted where abrupt changes occur in the activity levels. These depths may denote that dunes of that mean amplitude were present during part of the flow event. The duration of flow involving dunes of that mean amplitude, however, is also reflected by that depth (*3*).

Obviously, any one type of sediment transport may be partially or totally masked by other types, resulting in an overall remaining distribution of activity that cannot be readily separated into its various components. The crux of the experiment then is found in the interpretation of the vertical activity distribution within the injection strip. Hydrographic records and changes in bed configuration determined by soundings, e.g., utilizing the Dual Channel Stream Monitor, would assist in evaluating and interpreting the final activity distributions. In any event, the estimation of d will be difficult; in many situations it will be highly speculative.

Summary

In this investigation the radioisotope scandium-46 was utilized to measure the sediment transport velocity and sediment discharge of 0.35- to 0.297-mm. sand in a small laboratory flume. Because a large part of this study was devoted to a review of previous sediment trans-

port investigations utilizing radioactive tracers, a summary of the literature review is given in addition to the specific experiment with scandium-46.

Review of Literature

Labeling of sediments for transport investigations has been accomplished on both natural and artificial sediments. Natural sediments, or their primary constituents, have been irradiated in slow neutron fluxes or permitted to sorb the desired radioisotope. Artificial tracers, e.g., glasses, have been prepared by fixing the desired element in the glass, crushing, and sieving to obtain the required size distribution followed by irradiation to produce the tracer material. Incorporation of the radioisotope prior to grinding has also been performed, but this requires special handling facilities and poses a greater health hazard to the personnel involved.

The choice of a suitable radioisotope for the tracer depends upon a number of factors. It is the consensus of previous investigators (primarily concerned with the movement of sediments in estuaries and on the ocean floor) that the isotope should emit strong gamma radiations with energy levels near 1.0 Mev. This point has been stressed, because it is the only radiation capable of penetrating any measurable depths of sediment and water.

The isotope must possess a half life appropriate to the duration of the investigation. Selection will also depend on the method of labeling to be used. If the element is to be fixed in glass with subsequent activation in a slow neutron flux, then consideration must be given to (1) its ease of incorporation in the glass and (2) its ease of activation, i.e., its cross sectional capture of slow neutrons in the reactor. On the other hand, if the material is to be sorbed by the natural sediment, it is necessary to consider the permanency of fixation in the test environment. In any case, its hazard to the public must be carefully considered.

Sediment transport investigations have been largely concerned with the redistribution of tracer material placed on or near the bottom of seas and estuaries. Emphasis has been placed on the direction of sediment transport, the rate of dispersion of sediment, and the path of sediment movement as defined by wave action or tidal currents. The results of such investigations have been largely qualitative in nature. The only exception has been the work of the Hydraulics Research Station in Wallingford, England, under conditions of equilibrium flow in a large laboratory flume.

The measurement of radioactive tracers is discussed in terms of the different detector systems available. The principal advantage of the differential scintillation spectrometer compared to the Geiger-Mueller system is its ability to accept only those pulses corresponding to the characteristic gamma photon energies of the particular radioisotope. In addition, the spectrometer permits (1) the rejection of already existing activity, i.e., background, having emission energies different from the tracer, (2) rejection of scattered radiation from attenuation of the photons in passing through matter, (3) use of different tracers simultaneously, i.e., radioisotopes having well-defined energy spectrums whose primary photon energies can be separated by pulse height discrimination, (4) the accurate measurement of high levels of activity without loss of counts from "dead" time of the system, and (5) within limits the calibration of the detector with regard to spatial distribution of the tracer.

Laboratory Flume Experiments

A preliminary investigation of sediment transport was conducted in a small laboratory flume. Scandium-46, a gamma emitter with a half life of 85 days, was employed. Natural sand (0.35 to 0.297 mm. in diameter) was labeled by the sorption technique. Ten grams (roughly 10^5 particles) of labeled sand was placed in a narrow band on the surface of the channel bed (50 feet long by 1.16 feet wide). The channel bed was subjected to a constant water discharge of 0.0148 c.f.s. for 5 days. No sediment was added during the experiment. In situ measurements of the tracer movement, made with a differential scintillation spectrometer, provided a semiquantitative estimate of the tracer distribution for various flow-time intervals. Vertical and lateral distributions of the tracer were qualitatively determined at various cross sections of the channel. Tracer sand was buried to a depth of 1.2 inches for the flow condition imposed upon the bed.

The attenuation of the gamma photons in passing through the sand were measured by interposing various thicknesses of sand between a plane source of scandium-46 and the detector (scintillation probe). The total linear absorption coefficient (μ_0) was calculated to be 0.138/cm. Corrections for attenuation of the buried tracer were made for the 125-hour activity distribution survey. An average sediment transport velocity of 0.0688 ft./hr. was calculated for the 0.350- to 0.297-mm. fraction of sand. The sediment discharge for this fraction was calculated to be 0.062 lb./hr. Measured sediment discharge of 0.074 lb./hr. was determined for the first 30 hours of flow.

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APPLICATION OF RADIOACTIVE TRACERS IN THE STUDY OF SEDIMENT MOVEMENT

[Paper No. 61]

By D. W. HUBBELL and W. W. SAYRE, *engineers, U.S. Geological Survey*

Abstract

Radioactive tracer techniques were employed in order to investigate the dispersion and transport of bed material in a test reach of the North Loup River near Purdum, Nebr. Sand particles, labeled with iridium-192, were used as tracers to enable observation of the natural dispersion and transport processes.

The amount of radioactivity and the number of tracer particles required for the experiment were determined by considering the sensitivity of the radiation-detection system, the characteristics of the test reach, and radiological safety factors.

In the experiment, the tracer particles were released from a line source that extended across the bed of the stream. As the tracer particles were transported and dispersed downstream, their longitudinal and lateral distributions in the bed were observed by periodic surveys with a sled-mounted scintillation detector, and their vertical distribution in the bed was observed by monitoring core samples. Information obtained from a laboratory calibration of the radiation-detection system under simulated field conditions was used to reduce the field data to a set of tracer-particle concentration-distribution curves.

The results of the field study indicate a potential for the wide application of radioactive tracer in sediment studies.

Introduction

One relatively new measuring technique that is being applied more and more frequently in studies of sediment transport is the radioactive tracer technique. This technique is essentially different from more conventional methods in that the displacement of individual particles or groups of particles is measured rather than such characteristics as the flux of sediment particles past a cross section, the net volumetric change in a reach, or the displacement of channel boundaries. As a result, the radioactive tracer technique is particularly useful in studies of the dispersion of sediment, the rate of travel of sediment particles, and in other studies concerned with the fundamental mechanics of sediment movement.

Although radioactive tracers can be extremely useful, a number of factors must be considered in their application. In general, the accuracy of measurements increases as the level of radioactivity increases. However, the higher the level of radioactivity, the greater is the possibility for harmful contamination and radiation exposure. As a result, in order to achieve a satisfactory balance between the level of accuracy and the level of radioactivity, all aspects of an experiment must be considered in detail. The fundamental factors for consideration include (1) the kinds and required accuracy of measurements to be made, (2) the radionuclide to be used as a tracer and the means for labeling the sediment particles, (3) the amount of the tracer to be used, (4) the method of introduction of the tracer into the flow system, (5) the method of detection of the radioactivity, and (6) the reduction of the data.

This paper discusses briefly some of these factors as they were considered in the design and implementation of a field experiment on the North Loup River, Nebr., that was conducted by the U.S. Geological Survey in November 1960.

Experimental Design

The primary object of the field experiment was to study the dispersion and transport of bed-material particles in a natural stream. Hence, it was decided to release radioactive tracer particles as a line source and then to monitor the longitudinal distribution of the particles at various times after release. The site on the North Loup River near Purdum, Nebr., was selected because of favorable experimental conditions. In particular, the North Loup River, which is in the sand hill region of north-central Nebraska, maintains a relatively constant water discharge of about 250 c.f.s. for prolonged periods of time; draws from, rather than contributes to, the ground water; has a bed normally composed of large dunes approximately 1.5 ft. high and 15 ft. long; and has bed material having a median diameter of about 0.29 mm. The specific reach used for the experiment is a man-made cutoff about 1,800 ft. long and 50 ft. wide that is fairly straight except for two minor bends, has stable banks, and is isolated from any community.

It was concluded that optimum results would be obtained by using a single, narrow-size range of tracer particles having a median fall diameter that was slightly coarser than that of the bed material. Such a size range was selected to insure that the tracer particles would not be in suspension any significant part of the time, but that the particles in the size range containing the greatest proportion of the bed material

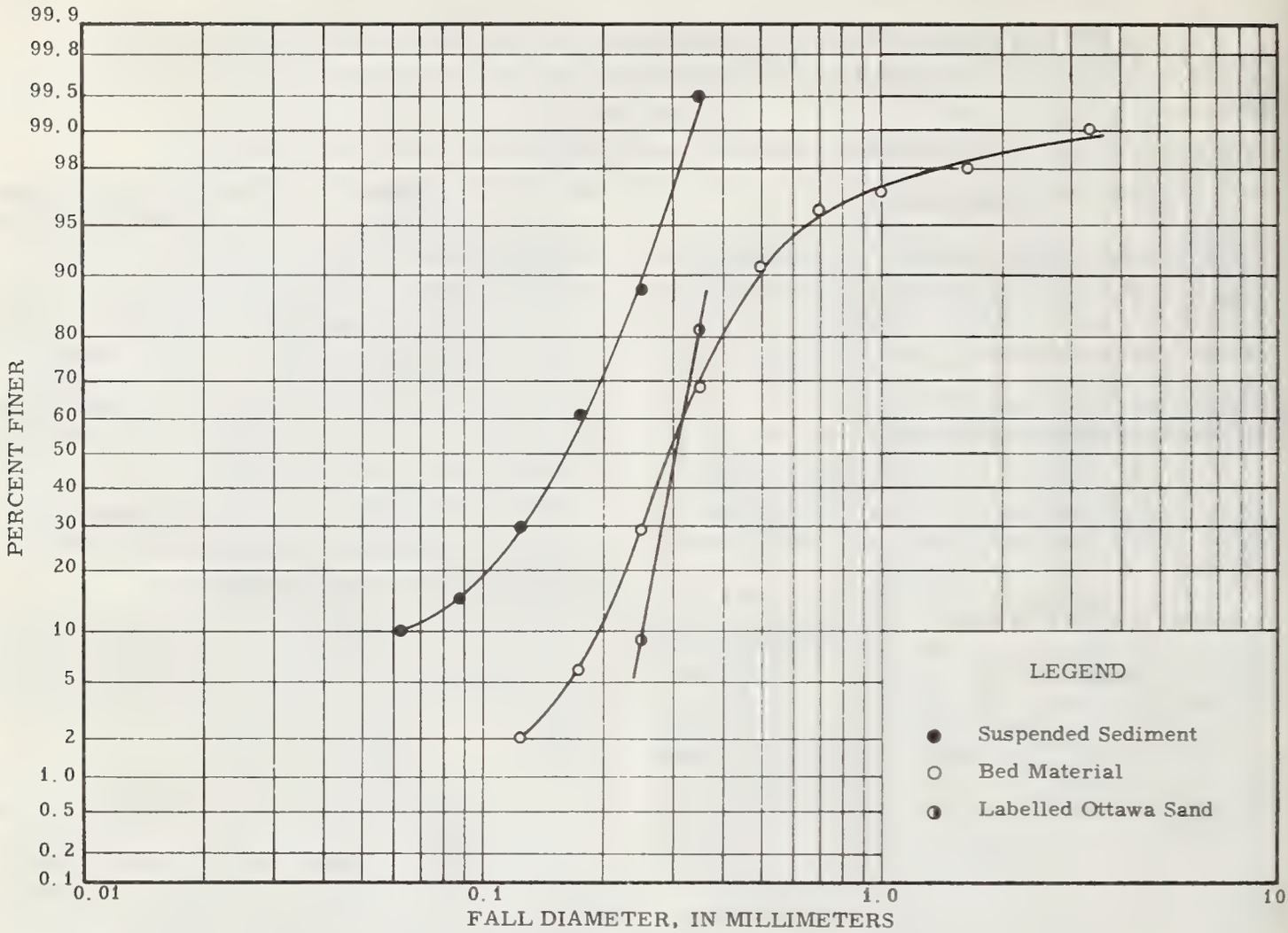

FIGURE 1. — Size distributions of measured suspended sediment, labeled Ottawa sand, and bed material.

would be represented. The selected tracer particles had a median particle size of 0.305 mm. and a distribution as illustrated in figure 1.

A comparison of the distribution of the tracer particles with those of the suspended sediment and the bed material, which are also presented in figure 1, shows that most of the tracer particles could have been in suspension at one time or another but that suspension probably occurred only infrequently.

Because the tracer particles would move primarily as bedload, in situ measurements seemed to be the most feasible means of defining the longitudinal dispersion of the particles. Such measurements virtually necessitated the use of a gamma-emitting radionuclide, because the penetration range of alpha and beta radiation is limited to only a few centimeters in water. A scintillation detection system was selected to monitor the gamma radiation because of its versatility and its inherently high sensitivity. The system consisted of a scintillation detector, pulse-height analyzer, count ratemeter, strip-chart recorder, and scaler. With this system, the number of gamma rays (photons) that interact with the crystal in the detector can be counted and displayed as a count rate or a count accumulation. Inasmuch as the number of gamma rays emitted from a source is directly proportional to the amount of radioactivity within statistical limitations and provided all other factors are the same, the count rate also is proportional to the amount of activity. Hence, if the amount of activity on each particle is proportional to the particle weight, measured count rates indicate the weight of tracer particles near the detector, provided, of course, suitable calibrations have been made.

For the experiment, the scintillation detector was housed in a watertight aluminum casing and mounted on a sled (fig. 2) that was dragged along the streambed. The detector was attached to the sled by a four-bar parallelogram linkage and supported on the bed by a wooden dish. This system allowed the detector to remain in continuous contact with the streambed during

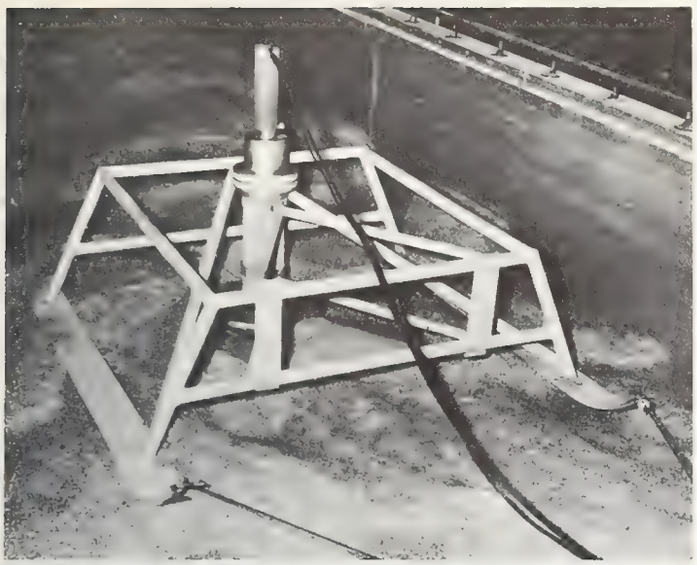

FIGURE 2. — Sled and scintillation detector. Note the parallelogram linkage that allows the detector to move independently of the sled and the wooden dish-shaped piece that supports the detector on the bed.

transport and to move vertically independently of the sled.

Iridium-192 was selected as the tracer because it (1) emits gamma rays having energies that are readily detectable underwater (0.14 to 0.90 Mev) ; (2) has a half-life of 74 days, hence, it would decay slowly enough to be readily detectable during the entire experiment, but rapidly enough to preclude long-term contamination of the stream; and (3) could be procured commercially already plated onto sand particles. The plating process consisted of wetting the sand in a solution containing the iridium-192 and then baking the sand at 700° F. for several hours. Tests showed that with continuous agitation of the particles immersed in water approximately 25 percent of the activity was abraded from the particles in 24 hours. Although it is impossible to relate the amount of abrasion the particles received in the tests to that likely to occur in a natural stream, 24 hours of agitation no doubt is equivalent to a relatively long time. Krone [1] also plated the radionuclide onto the sediment by an adsorption process. However, other methods of producing tracer particles have been used. For instance, Inose and others [2] incorporated the radionuclide into glass particles that simulated the sediment, and Lean and Crickmore [3] irradiated natural sand in a nuclear reactor. A summary of the various labeling techniques that have been used is given by Feely and others.[4] One important factor in connection with sand plated with a radionuclide is that the measured activity is proportional to the surface area of the particle rather than to its weight. If a wide range of particle sizes is labeled, this factor must be considered in converting a measured count rate to a weight of tracer particles; however, if the particles are relatively uniform in size, the count rate is also approximately proportional to particle weight.

The amount of radioactivity required for the experiment was determined by considering the natural background radiation, the volume of sand throughout which the particles would be dispersed, the decay rate of the activity, the adsorption characteristics of the sand and water, the efficiency of the detection system, and the geometrical orientation of the detector to the tracer particles. The effects of the latter three items can be characterized by the "sensitivity" of the detection system, which is the count rate per unit of activity per unit volume under specific experimental conditions. The sensitivity of a detection system to a uniformly distributed source such as sand tracer particles can be estimated by defining a count-rate attenuation function through measurements of the count rate from a weak point source buried at different locations and distances relative to the detector and then integrating the attenuation function over the volume through which the tracer particles will be distributed.

Once the sensitivity is determined, the amount of radioactivity (M) required for an experiment can be computed from

$$M=\frac{(R_o-R_b)\ V\ e^{0.693t/T}}{S}$$

in which

R_o-R_b is the minimum net counting rate over background that is required during the experiment for statistical significance.

V is the estimated volume through which the tracer particles will be dispersed at the end of the experiment.

S is the sensitivity of the detection system for the conditions of the experiment.

$e^{0.693t/T}$ is a correction factor for radioactive decay.

[1] KRONE, R. B. FIRST ANNUAL PROGRESS REPORT ON THE SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. Calif. Univ. Inst. Engin. Res. 118 pp., illus. 1957.

[2] INOSE, S., KATO, M., SATO, S., and SHIRAISHI, N. THE FIELD EXPERIMENT OF LITTORAL DRIFT USING RADIOACTIVE GLASS SAND. 1st U.N. Internatl. Conf. on the Peaceful Uses of Atomic Energy Proc. 15: 211-219, illus. 1956.

[3] LEAN, G. H., and CRICKMORE, M. J. THE LABORATORY MEASUREMENT OF SAND TRANSPORT USING RADIOACTIVE TRACERS. Dept. Sci. and Indus. Res., Hydraul. Res. Sta., Wallingford, England, 26 pp., illus. 1960.

[4] FEELY, H. W., WALTON, A., BARNETT, C. R., and BAZAN, F. THE POTENTIAL APPLICATIONS OF RADIOISOTOPE TECHNIQUES TO WATER RESOURCE INVESTIGATIONS AND UTILIZATION. Atomic Energy Comm. Res. and Devlpmt. Rpt. NYO 9040, 340 pp. 1961.

where

t is the duration of the experiment.
T is the half-life of the radionuclide.

In the design of this experiment, a net minimum counting rate equal to one-half of the background rate was considered acceptable for the condition of a uniform distribution of tracer particles throughout the test reach. By assuming that the experiment would last for one month, it was determined that 40 millicuries of iridium-192 would be required.

When radioactivity is labeled on sediment particles, each particle is a separate source and the count rate measured by the detection system depends not only on the amount of radioactivity, but also on the distribution of the tracer particles. As a result, sufficient particles must be used in order to minimize the possibility of significantly nonuniform distributions within the bed. If the distribution of tracer particles throughout the bed is random, as the concentration of tracer particles increases, the relative distribution of the tracer particles tends to become more even and random variations in the number of tracer particles per unit volume of bed material tend to have less effect on the count rate.

The random variations in the number of particles in a given volume can be characterized by the coefficient of variation or the relative standard deviation. If the variation in the number of tracer particles in a given volume is assumed to follow the Poisson distribution, the coefficient is $100/\sqrt{N}$, where N is the mean number of tracer particles in the given volume of bed material. For experimental design, a coefficient of about ± 6 percent seems to be adequate when the volume associated with N is defined as that volume of the bed from which 50 percent of the measured gamma rays (counts) emanate when the sand bed contains a uniformly distributed source of infinite extent and N is taken as the required number of tracer particles within the volume at the end of the experiment when the tracer particles are distributed over the test reach.

The coefficient of variation provides only an index to the expected variation in count rate attributable to random variation in the number of tracer particles. However, if a relatively long ratemeter time constant is used and the detector is moved along the bed, the fluctuations in count rate due to local variations in the distribution of tracer particles are damped appreciably. Sensitivity measurements for the iridium-192 and the detection system used in the experiment indicated that 50 percent of the counts emanate from within 4.4 in. of the center of the 2-in. by 2-in. detector crystal, which is equivalent to a volume of 116 cu. in. The coefficient of variation with this volume for the total reach volume of $1,800 \times 50 \times 1.5$ cu. ft. and the 40 lb. of sand (approximately 4.6×10^8 particles) used in the experiment was 6.6 percent.

In order to establish the exact relationship between count rates and the weight of tracer particles, the detection system was calibrated under conditions that simulated the actual experimental environment. The calibration was made in a 4-ft. diameter by 4-ft. high tank in which known weights of tracer particles were mixed with known volumes of natural bed material similar to that in the North Loup River. Count rates were measured for different depths of uniformly distributed particles and for different ratios of the weight of tracer particles to the volume of natural sand (concentrations). In order to reduce the background count rates, a depth of 9 in. of water was maintained over the sand bed throughout the calibrations.

The results of the calibrations, which were for two different instrument settings, are shown in figure 3. This figure shows that the adjusted count rate, which is a count rate adjusted for radioactive decay, varies with depth as well as with concentration for all depths less than

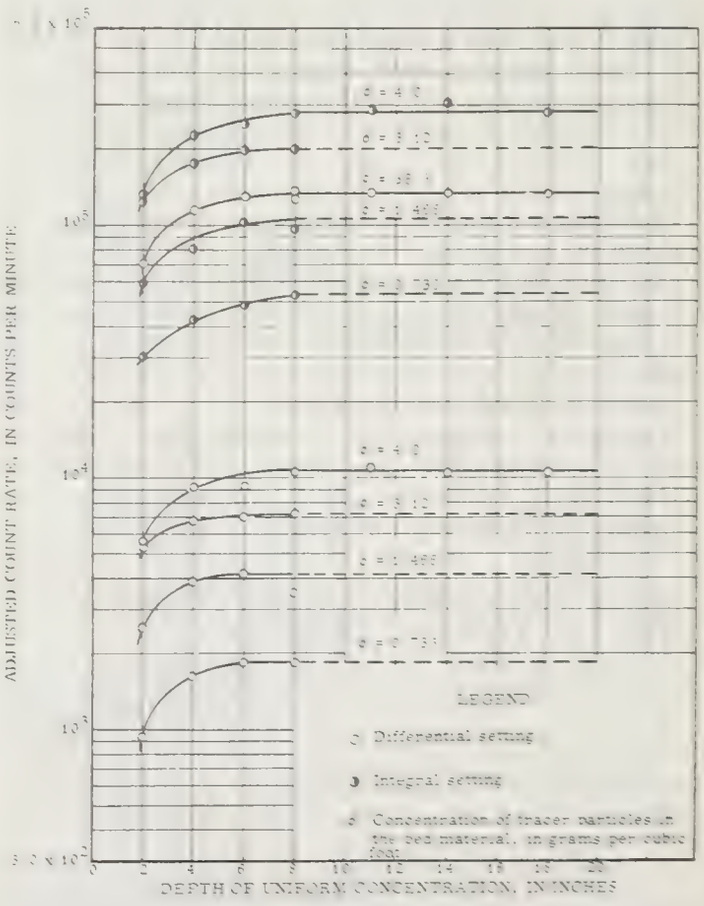
ADJUSTED COUNT RATE, IN COUNTS PER MINUTE
DEPTH OF UNIFORM CONCENTRATION, IN INCHES
LEGEND
Differential setting
Integral setting
Concentration of tracer particles in the bed material, in grams per cubic foot

FIGURE 3. — Variation in adjusted count rate with the depth to which different concentrations of tracer particles are uniformly mixed.

about 8 in. For depths greater than about 8 in., the adjusted count rate varies only with concentration. In the experiment it was anticipated and eventually borne out that the particles would be distributed to depths greater than 8 in. As a result, the only part of the calibration curves required for converting the count rates observed in the field to concentrations were those parts for an 8-in. depth.

Data Collection and Results

In order to study longitudinal dispersion, the tracer particles were introduced essentially as a line source by depositing the 40 lb. of tracer particles in 2-lb. lots at 2-ft. intervals across the width of the channel. Each 2-lb. lot was labeled with 2 millicuries of iridium-192. The labeled particles were placed on the streambed by use of an apparatus consisting of an electric can opener and a movable funnel tube that were mounted to the stern of the boat. In the placement (dosing) operation, the funnel tube was lowered until the bell at the bottom of the tube rested on the streambed, then a can containing the tracer particles was opened with the can opener and poured down the funnel tube to the streambed.

About 2 hours after the dosing operation was completed the first longitudinal traverse was made. Just before this traverse, as well as all subsequent ones, the detector sled was placed by hand upstream from the dosing section and the natural background radiation was recorded for about 2 minutes. After background counting, the boat was released and maneuvered downstream. The longitudinal traverses were made with the boat and the detector sled arranged in tandem as is shown in figure 4. The

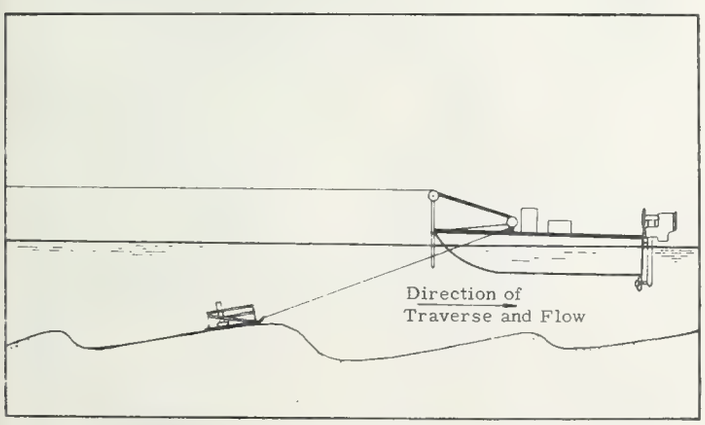

FIGURE 4. — Arrangement of boat and sled for the longitudinal traverses.

boat, which faced upstream and pulled the detector sled, was maneuvered downstream by means of an outboard motor having reverse controls. Mounted on the front end of the boat was a distance-measuring cable and reel that functioned both as a stay line and as the distance marking system. In operation, the cable, which was fixed at the upstream end of each segment of the test reach, was unwound manually so that the boat and sled moved downstream at a uniform controlled rate. When buttons that were located at definite intervals along the cable tripped an event-marking switch, ticks were made on the recorder charts. In this way the recorder charts were provided directly with a distance coordinate.

The tracer particles were tracked daily by making longitudinal traverses with the radiation detection equipment down the left and right sides of the stream along paths that were roughly one-third of the channel width away from the bank. Typical data produced by the longitudinal traverses are shown in figures 5 and 6. The distribution curves in these figures were established by adjusting the observed count rates for radioactive decay and then converting the adjusted count rate to a concentration with the calibration curves. One particularly interesting occurrence is indicated by the distribution curve in figure 6. This curve shows that on about the sixth day after dosing, a relatively large number of tracer particles appeared just downstream from the dosing section along the left side of the stream. Apparently, tracer particles were trapped in a deep trough and remained buried beyond the range of the radiation detection equipment for several days. About the sixth day after dosing, the trapped particles were released by a train of dunes having deep troughs. The particles then began to disperse in a normal manner. As a result of the temporary storage, the distribution curves for the left side of the test reach were less indicative of the general distribution that applies for the average flow condition than those for the right side.

In addition to the longitudinal traverses, several traverses were made to define the lateral distribution of the tracer particles. Figure 7 shows the lateral distribution of the particles 185 ft. downstream from the source 2 days after dosing and 415 ft. downstream from the source 5 days after dosing. The deficiency of particles along the left bank near the source is evident in the distribution for the second day after dosing. The distribution curve for the fifth day after dosing indicates a reasonable uniformity of the tracer particles.

In order to develop accurate concentration-distribution curves from the observed count rates and the calibration curves, it was essential that the depth to which particles were dispersed generally was greater than 8 in. and that no general vertical concentration gradient per-

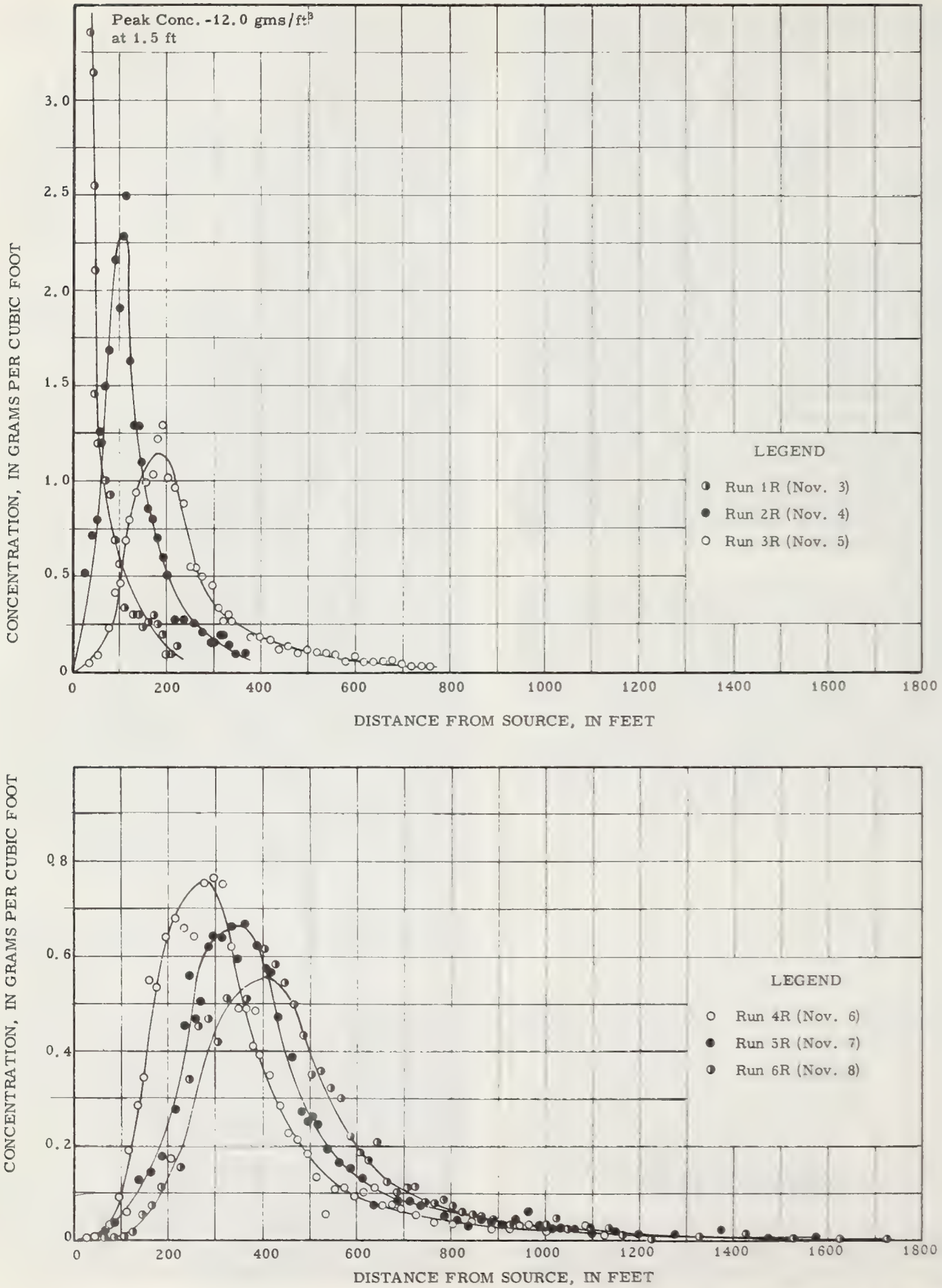

FIGURE 5. — Longitudinal distribution of labeled particles along the right side of the channel, Nov. 3 to 8, 1960.

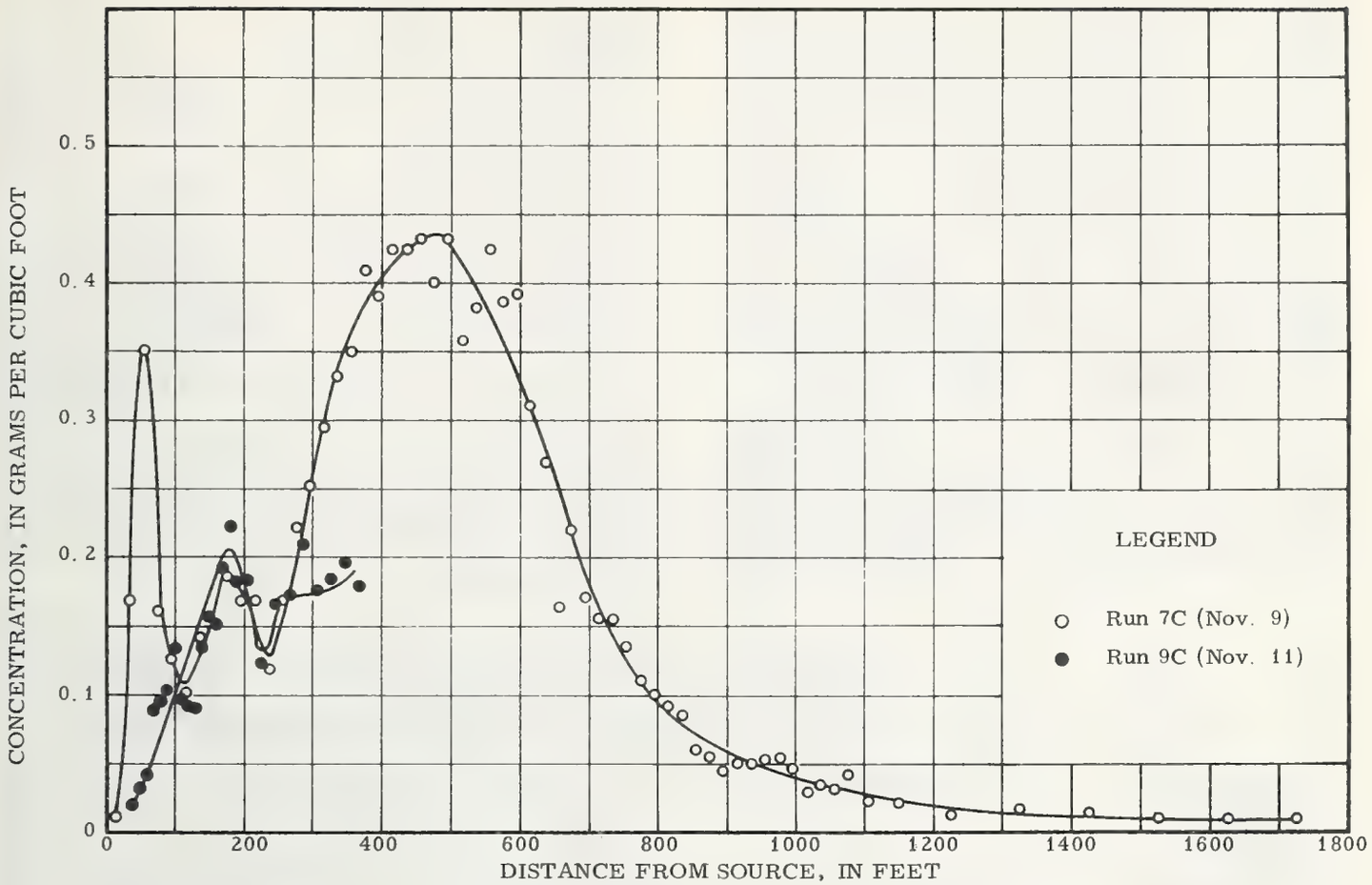

FIGURE 6.—Longitudinal distribution of labeled particles along the center of the channel, Nov. 7 and 9, 1960.

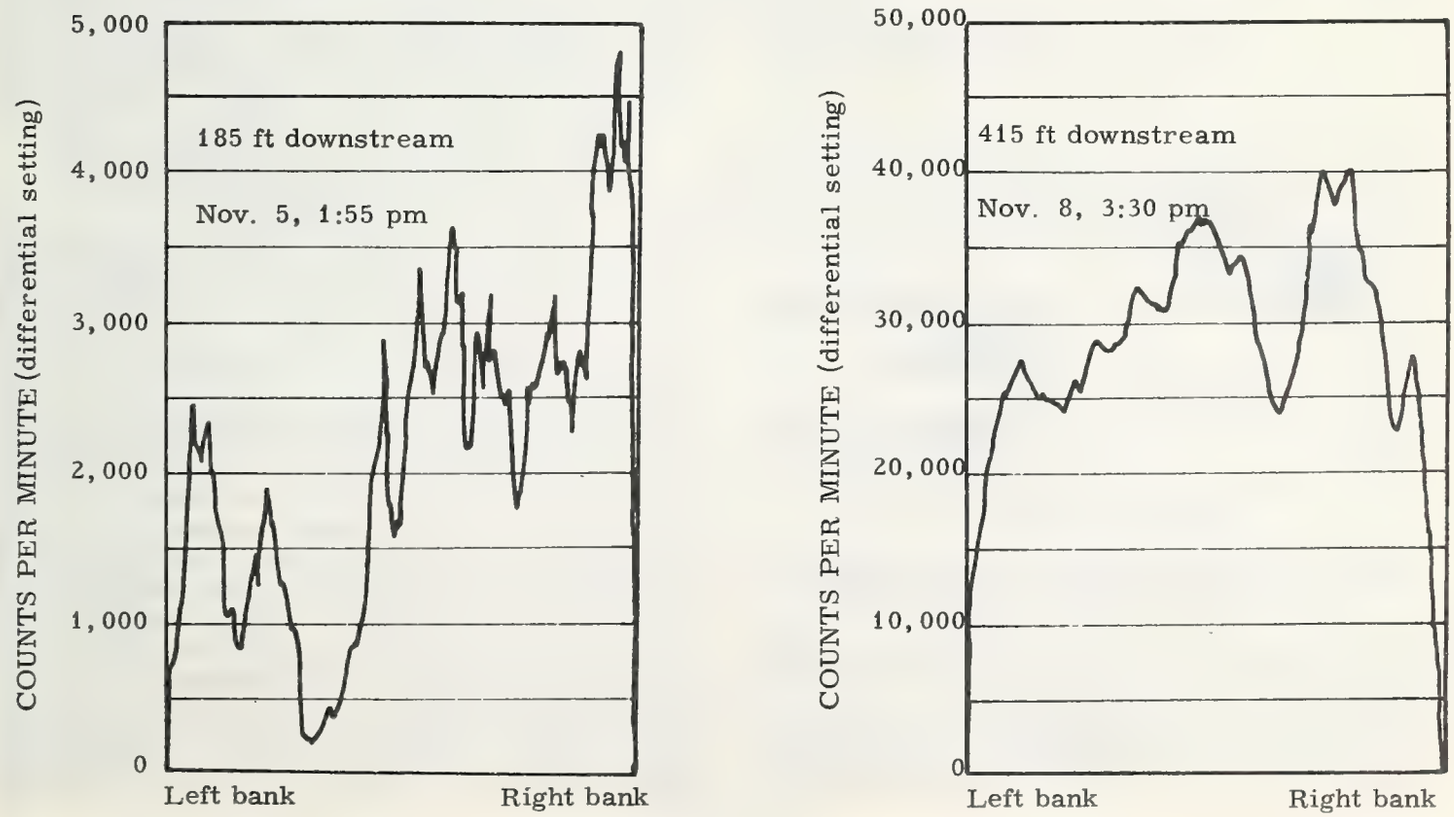

FIGURE 7.—Lateral distribution of labeled particles downstream from the source on Nov. 5 and 8, 1960.

sisted throughout the streambed. Two different methods were used to verify the fact that these two conditions existed. The first method consisted of defining the vertical distribution of tracer particles into the bed. This was done by collecting core samples of the bed material at various lateral and longitudinal positions in the test reach. The core samples were collected with a 1½-in. diameter sampler capable of withdrawing a 3-ft. long core. For analysis, the cores were ejected in 2-in. increments with a hand jack attached to the end of the sampler plunger and each increment was counted separately with a scaler by using the scintillation detector and sled in an inverted position.

Some typical observed vertical distributions are shown in figure 8. A common characteristic of all of the vertical distributions was that the count rate was highly variable with depth into the bed and no semblance of a continuous vertical distribution pattern was definable. Probably

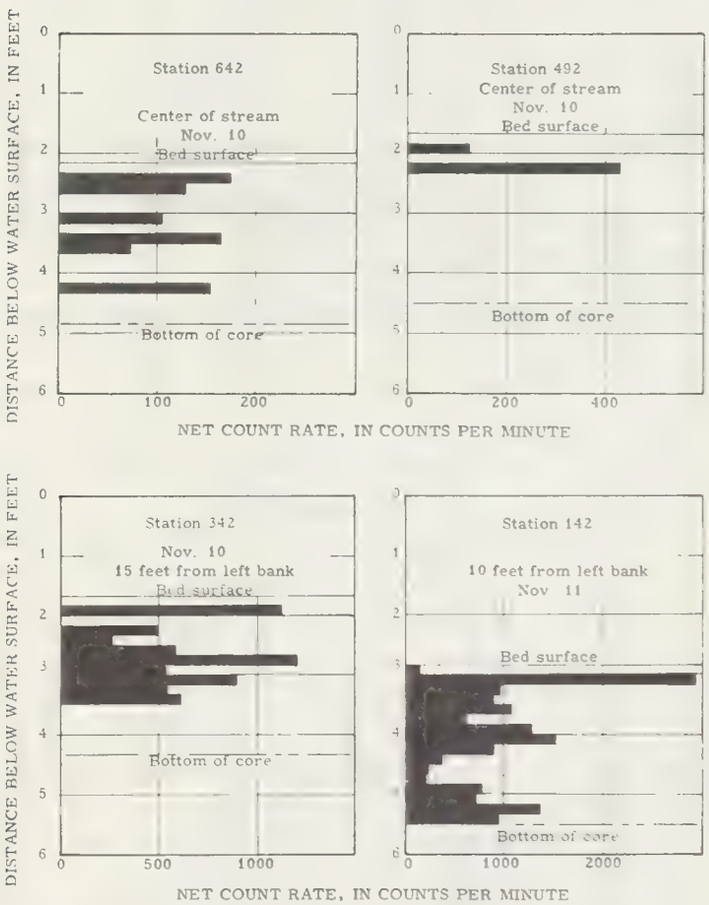

FIGURE 8. — Vertical distribution of labeled particles at selected verticals on Nov. 10 and 11, 1960.

[5] KARAKI, S. S., GRAY, E. E., and COLLINS, J. DUAL CHANNEL STREAM MONITOR. Amer. Soc. Civil Engin. Proc. 87, No. HY6: 1-16, illus. 1961.

[6] HUBBELL, D. W., and HAUSHILD, W. L. DISCUSSION OF "DUAL CHANNEL STREAM MONITOR." Amer. Soc. Civil Engin. Proc. 88, No. HY4: 287-291, illus. 1962.

the variations resulted largely from the passage of dunes having different amplitudes, mean elevations, and concentrations of tracer particles. Unfortunately, as a result of these irregularities of the movements of the dunes, neither the distance below the water surface, distance below the bed surface, nor depth of activity could be used to normalize the vertical distribution of count rate. Hence, the distributions from the separate verticals could not be combined to provide an average gradient. However, the distributions from the separate verticals were combined to show that the average depth of the zone through which the sediment particles moved was about 1.45 ft. Because the average depth of the zone of movement was considerably greater than 8 in. and no discernible vertical distribution pattern appeared to exist, it seems reasonable to assume that the observed longitudinal and lateral distributions generally were representative of the actual distribution of the tracer particles.

The validity of converting the observed count rates to concentration of tracer particles with the calibration curves for an 8-in. depth was further verified by comparing the distribution of observed count rate with records of the bed configuration. The comparisons showed that no correlation existed between the depth of the zone of particle movement, as characterized by a profile of the bed configuration over which the detector passed, and the observed count rate. The apparatus for measuring bed configuration was a dual-channel ultrasonic depth sounder,[5] which was mounted on the bow of the boat with one transducer facing upward and another facing downward so that both the water- and bed-surface profiles were recorded. A typical record of the bed configuration is shown in figure 9.[6]

Conclusions and Application of Results

The results of the field study and subsequent laboratory flume studies indicate that the concentration of labeled sediment particles at any point in the streambed at any time can be determined conveniently, accurately, and safely with radioactive tracer techniques. This conclusion is important, because it implies the feasibility of determining, by experiment, distributions of labeled particles in a stream channel with respect to space and/or time for a wide variety of conditions. For example, by selective labeling, certain aspects of the behavior and distribution of particles having definite sizes, shapes, specific gravities, and other characteristics can be determined. Furthermore, by using a scintillation detector in conjunction with a pulse-height analyzer, different radionuclides can be distinguished from one another so that

several selected labels can be used simultaneously to observe the behavior of different types of particles in the same experiment. For example, the authors currently are conducting laboratory flume experiments in which three different radionuclides are used to trace, simultaneously, the movement of three different sizes of particles in a bed material of naturally graded sand. Measurement of concentration distributions of labeled particles will provide new insights into the phenomenon of sediment transport and will supply the kind of data necessary for analyzing sediment transport as a random phenomenon that can be characterized by probability laws and treated as a stochastic process.

In addition, observed longitudinal distributions of tracer particles released from a uniformly distributed source provide a means for computing sediment discharge directly with a continuity-type equation. The computation is based on the idea that (1) the time rate of movement of the mean of the distribution is equivalent to the average particle velocity and (2) the extent of the tracer particle distribution within the bed defines the effective cross-sectional area through which the particles move.

The examples given above indicate only a few of the many possible applications of radioactive

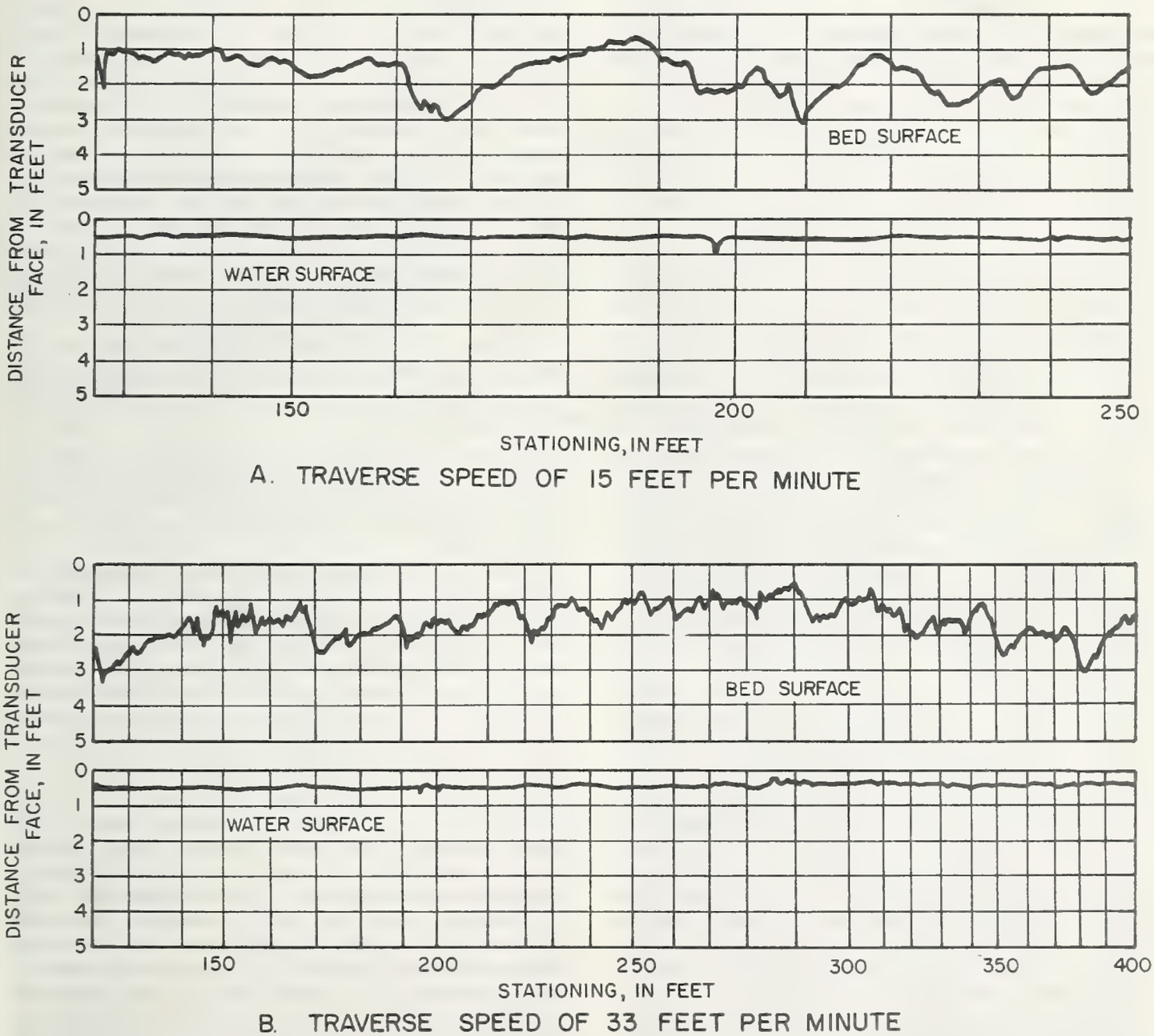

FIGURE 9. — Water- and bed-surface profiles defined by the dual-channel stream monitor. (From Hubbell and Haushild, 1962.)

tracers in sediment transport research. Probably only a small part of potential value of radioisotope techniques in sediment research has been realized. It is the authors' hope that this paper will serve to assist others with the development of new applications.

USE OF RADIOISOTOPES IN SEDIMENT TRANSPORT STUDIES

[Paper No. 62]

By R. S. CUMMINS, JR., *engineer*, and L. F. INGRAM, *research physicist, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.*

Synopsis

In 1956 the U.S. Army Engineer District, San Francisco, in conjunction with the University of California, undertook a sediment tracing project using radioactive tracers in San Francisco Bay. This initial investigation indicated the tremendous possibilities of the technique for sediment studies and assisted with the development of special tracing equipment and methods of labeling sediments. In 1960 the U.S. Army Engineer Waterways Experiment Station selected a group of trained personnel to continue such tests for other Corps of Engineers offices as the occasion arose. Tests have since been conducted at Galveston Harbor, Tex., and in Cape Fear River, south of Wilmington, N. C. The tracer material selected for use in each instance was gold^{198}, which is a short half-life, low energy gamma emitter. The labeling methods employed, however, were different in that the Galveston test material was a manufactured particulate tracer with metallic gold fused in glass. The Cape Fear test material was existing sediment with the radioisotope adsorbed on it. The labeling procedure in the Cape Fear test involved the introduction of the isotope into a slurry of natural sediment. The slurry was agitated, which allowed the tracer to be adsorbed on the particles. The mixture was then released in a uniform pattern at the predetermined placement site. No problems other than those inherent in the handling of radioactive material and contamination of tracing equipment were encountered.

Surveying operations, both background and postplacement, were conducted, using either transits or sextants for position fixing at time intervals to permit correlation of location and activity. A continuous recording of bottom activity was made during the life of the tracer.

The results obtained were plotted on charts that showed either development patterns of movement between daily surveys or deposition areas for transported sediment. Since the labeled material may be lost, either by being covered or by being widely dispersed, quantitative measurements are extremely difficult. Limitations are imposed by (1) rapid rate of movement required because of the short-lived isotope; (2) difficulty of surveying in areas containing wreckage or debris; and (3) inability to measure labeled material in suspension or in flocs.

Introduction

This paper is not intended to be a treatise on the subject of radioisotopes, but rather a brief discussion of the Corps of Engineers efforts with the use of radioisotopes for sediment tracing. In 1956 the U.S. Army Engineer District, San Francisco, entered into a contract with the University of California to develop techniques and equipment suitable for use in connection with a comprehensive investigation of the sedimentation and shoaling processes in San Francisco Bay. From this effort came a method of labeling sediment, the development of an underwater detector to trace sediment movement and other field equipment, handling procedures, and field techniques.

In 1960 the U.S. Army Engineer Waterways Experiment Station selected and trained a group to continue such tests for the Corps of Engineers as the occasion arose. Because of the specialized training and equipment involved, it was more practical to utilize a single group for tracer purposes than to train and equip groups in each District Office that might have use for tracer techniques.

Since establishment of the test group at the Vicksburg Waterways Experiment Station, tests have been conducted at Galveston Harbor, Tex., and in Cape Fear River, N. C. Additional tests are scheduled early in February at Galveston Harbor. In each instance the problem was to trace the movement of bottom sediment. Although the techniques employed were somewhat similar, the method of labeling the tracer material was different in each case.

The tests in Galveston Harbor were conducted in an effort to determine the path taken by resuspended sand that comes from a generally known source area. Galveston Harbor (fig. 1), which serves not only Galveston but also Houston and Texas City, is protected by stone jetties extending several thousand feet beyond the island itself. It was believed that material outside the North Jetty moved either through the jetty or around the end and into the ship channel, necessitating extensive dredging to

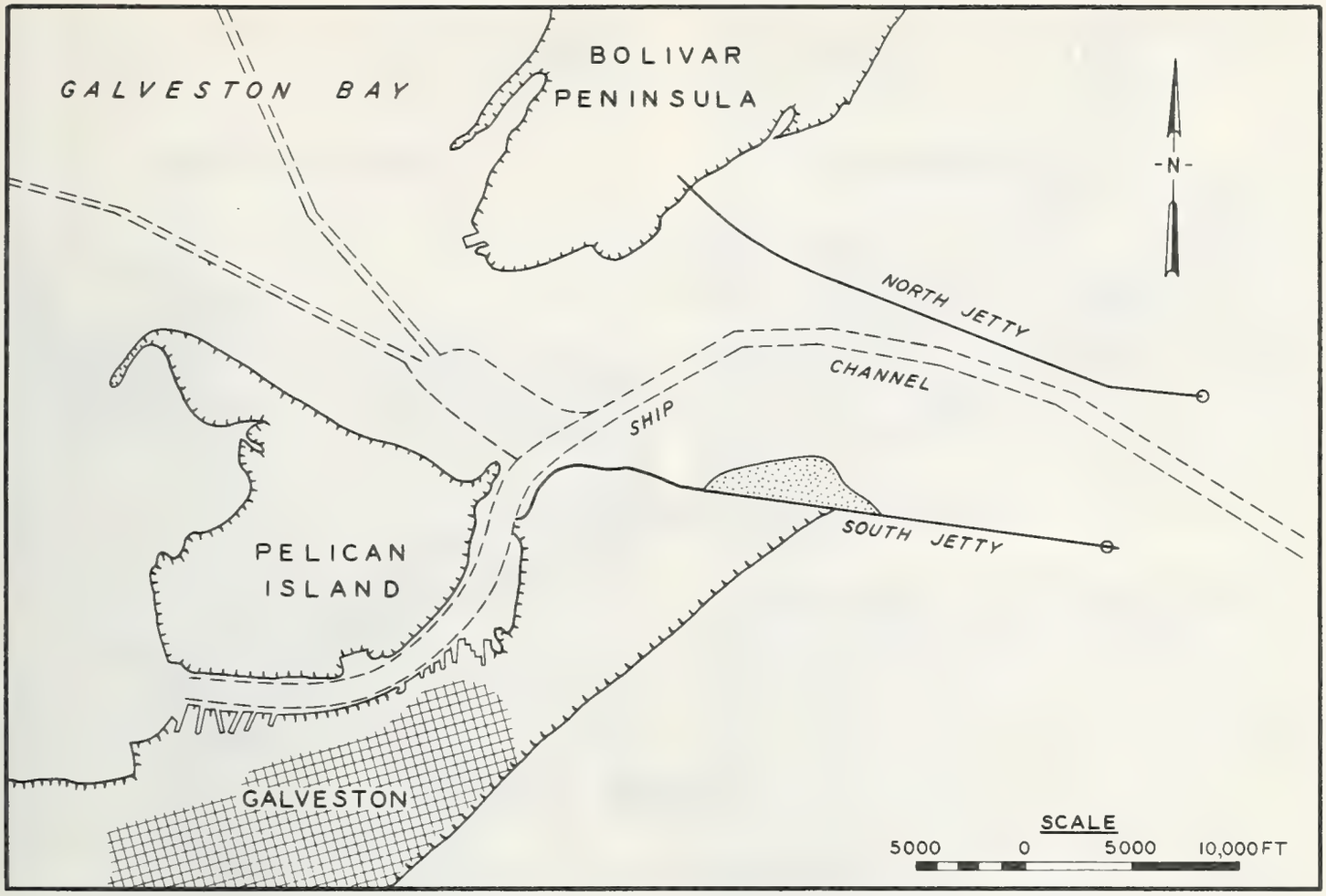

FIGURE 1. — Galveston Harbor and locations of stone jetties.

maintain channel depths. Therefore, the purpose of the North Jetty tests at Galveston, made during June 1961 and June 1962, was to trace the movement of material outside North Jetty to the shoaled areas within the ship channel. Similar tests are scheduled for the South Jetty. Three deposits of radioactive material were made in water depths of 3, 6, and 9 ft. during June 1961, and six deposits were made in deeper water during June 1962.

Tests were made in the Cape Fear River at the Sunny Point Army Terminal to determine the necessity of diking dredge spoil areas (fig. 2). The Army Terminal is located about 20 miles south of Wilmington, N.C. The entire wharf facility was dredged out of a shallow area and the spoil pumped across the river channel into a disposal area. Dock facilities consist of three wharves with 800-foot-wide basins connected by 300-foot-wide channels and served by three entrance channels. Shoaling in the wharf areas is extremely rapid and averages about 0.8 ft. of depth per month near the north wharf and about 0.6 ft. per month near the center wharf. Suspected source areas include not only the spoil areas in the vicinity but also those upstream from the terminal.

Prior to the actual conduct of tracer surveys, preliminary investigations are made of such items as velocity and current patterns in the vicinity of the problem area, bottom sediment samples, and dredging surveys. Such information is necessary to locate deposit areas, areas where surveying might be difficult or dependent on tide conditions, and areas containing wreckage that might hamper surveying operations.

License Requirements

Once it is decided that tracer techniques are feasible in a particular problem, the tracer material must be selected and a license obtained from the Atomic Energy Commission for procurement of the radioactive material and its use in a specified location. A separate license must be procured for each type of radioactive label and test location. The license request is filed through the Safety Division, Office of Chief of Engineers, and the office of Environmental Hygiene Agency of the U.S. Army Surgeon General. Standard forms are available for the pur-

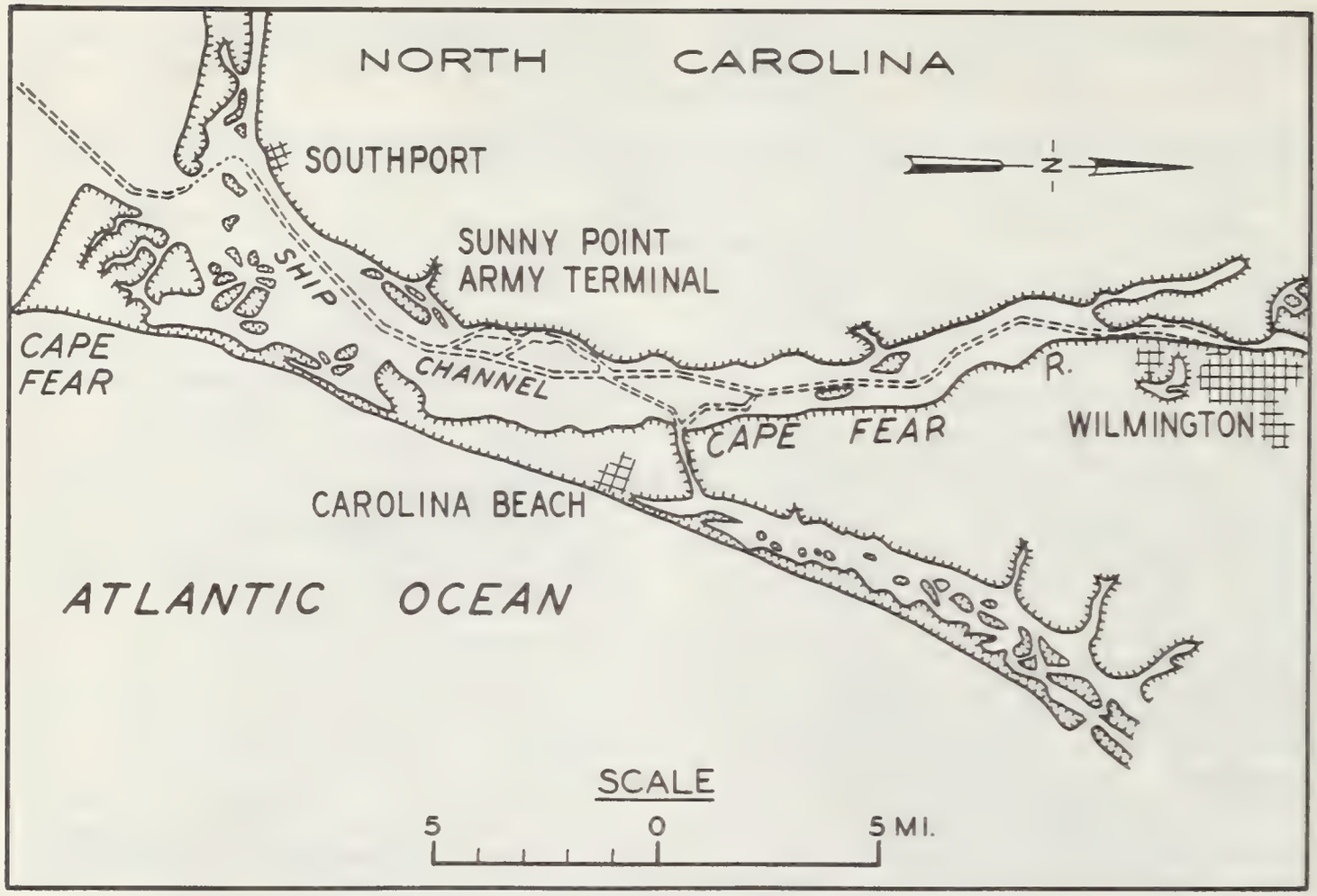

FIGURE 2. — Sunny Point Army Terminal, Cape Fear River.

pose. On these forms detailed steps of the entire operation must be defined. Information must be provided on the quantities of isotopes desired; handling, storage, and transportation; and safety precautions to eliminate possible health hazards to users and contamination of adjoining areas. To satisfy prelicense requirements, a public meeting with city, county, and State officials, representatives of the health department, and other interested agencies that might be affected must be held to explain the tests and obtain the approval of those concerned. Several public meetings were held in connection with the tests at Galveston and in Cape Fear River.

After the approving agencies are completely satisfied that there are no possible health hazards to the users or residents of the affected area, a specific license is granted for a period of 1 year. This license authorizes possession of a specified amount of a particular isotope for use as described in the license. Procurement of a license requires considerable lead time, but with experience in filing for such license, the time probably can be reduced. Increased experience of the licensee also helps decrease the lead time.

Selection of Isotope

The selection of the isotope to be used as the tracer is of considerable importance. Controlling factors are (1) duration of a single test or required half-life of the isotope to be used; (2) energy level of and type of emission desired from the isotope; and (3) the type of labeling procedure that will yield accurate data with regard to particle size. Consideration must be given to the fact that radioisotopes having significant half-lives are frequently elements required in the metabolic processes of marine organisms or animals. Also isotopes of higher energy create special handling problems and are not readily available in forms easily applied to various types of sediment.

If the entire sample of radioactive material is released into a flowing stream, the concentration of radioactive material cannot exceed the maximum permissible concentrations stated in the National Bureau of Standards Handbook No. 69.[1] For example, the maximum allowable concentration of gold ^{198} is given as 5×10^{-4} mi-

[1] U.S. NATIONAL BUREAU OF STANDARDS. MAXIMUM PERMISSIBLE BODY BURDENS AND MAXIMUM PERMISSIBLE CONCENTRATIONS OF RADIONUCLIDES IN AIR AND IN WATER FOR OCCUPATIONAL EXPOSURE. Handbook No. 69, 95 pp. 1963.

crocuries per cubic centimeter, or about 1.9 microcuries per gallon. The amount of tracer material required, however, must be sufficient to yield significant counting rates when the material is uniformly distributed over the problem area. The sensitivity of the detecting device governs the concentration required to yield a mathematically significant increase over the natural background count rate. An accepted significant increase under laboratory conditions, using a statistical nine-tenths error determination, is an increase of more than $1.645\sqrt{R}$, where R is the observed counting rate. (This value is the limit of error for a 0.10 level of significance.) In field tests, where background rates vary by as much as a factor of 10, it is not unrealistic to use the figure, $10\sqrt{R}$, as a significant value. The quantity of material to be labeled may be calculated by taking into consideration grain-size distribution, probability of deposition or movement, and activity obtainable per particle for the smaller particles.

The tracer material selected was gold^{198}, which is a low energy, short-life, gamma emitter. The energy of this isotope is spectral, that is, of nearly Gaussian distribution, at the photopeak of 0.41 Mev (million electron volts). The basis for selection of this particular isotope and tests of labeling sediment are discussed by Dr. Krone [2] in the first annual report.

Sediment Labeling

Two different labeling procedures have been successfully used at Galveston and in Cape Fear River. The bottom samples from Galveston Harbor were sand ranging in grain size from 10 to 140 microns, with a median size of about 40 microns. A manufactured glass containing 0.3 to 0.5 percent gold by weight was ground and graded to the proper size distribution to duplicate bottom sediments. The uniformity of samples taken from the source area permitted the use of such a method of tracer preparation. The manufactured glass was of such chemical composition that there were no long-lived isotopes present. It was necessary to add potassium in the glass to reduce the melting point to a practical value. Since potassium is a short-lived, high-energy isotope, the irradiated samples of glass were retained at Oak Ridge National Laboratories for 24 hours to permit decay of activity to maximum permissible shipping limits.

The bottom sediments in the Cape Fear River area were largely sandy silts, except for the fraction dredged from the Sunny Point Army Terminal, which was fine silt. This silty material was suitable for labeling by the adsorption method. The adsorption method of labeling is accomplished by adding a solution of gold ^{198} dissolved in aqua regia to a slurry containing the natural sediments. The mixture is thoroughly blended and allowed to remain in residence for a period determined by concentration. The gold metal is adsorbed onto the soil particles, thus labeling them. This method was developed and used by the University of California in San Francisco Bay.[3]

Handling procedures resulting from the different methods of labeling create special problems and necessitate the development of suitable handling equipment. The particular tracer used at Galveston was ground, graded, and sealed in plastic bottles prior to irradiation at Oak Ridge, Tenn. The irradiated material was shipped to the test site in special sealed containers. The problem of placement was one of opening the containers and releasing the sediment at the proper depth in the selected area. This was accomplished by using a long aluminum tube attached to a boat, with a puncturing and flushing device mounted on the lower end of the tube (fig. 3). This device consisted of a three-bladed

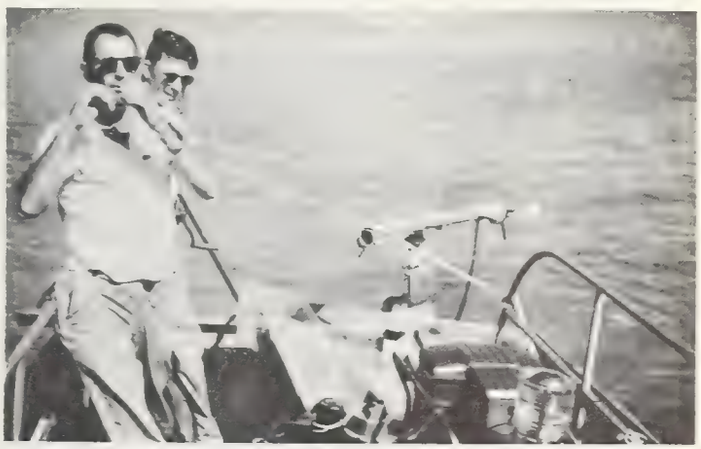

FIGURE 3. — Tracer release apparatus attached to survey boat.

spear that was fastened to a hollow shaft through which water was pumped into the punctured container (fig. 4). Annular holes in the bottom plate behind the spear allowed the mixture to flow from the container onto the harbor bottom. The plastic bottles were taken from the lead-

[2] KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. Calif. Univ. (Berkeley) Hydraul. Engin. Lab. and Sanitary Engin. Res. Lab. Prog. Rpt. 1. 1957.

[3] KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. SECOND ANNUAL PROGRESS REPORT. Calif. Univ. (Berkeley) Hydraul. Lab. and Sanitary Engin. Res. Lab. 1959.

KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. THIRD ANNUAL PROGRESS REPORT. Calif. Univ. (Berkeley) Hydraul. Lab. and Sanitary Engin. Res. Lab. 1960.

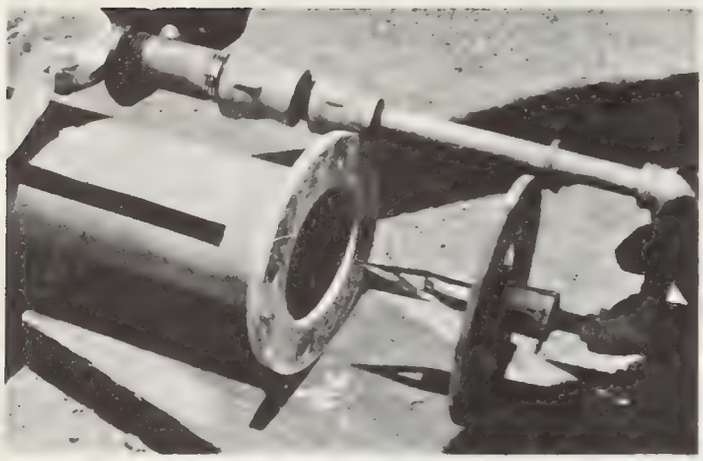

FIGURE 4. — Device for releasing sediment from container.

shielded shipping containers with 4-foot-long laboratory handling tongs, placed in the tube, forced down on the spear with a plunger, flushed, removed, and replaced in the shipping containers. In later tests during June 1962, aluminum containers were substituted for the plastic bottles as the plastic became brittle during the irradiation process.

The labeling of natural sediment, such as the silts and clays encountered in the Cape Fear River tests, required the use of a labeling vessel equipped with a mixing device, a method of introducing the tracer, and an ejection device, all of which had to be operated remotely. A cylindrical tank with truncated cone bottom was constructed (fig. 5). An electric motor drove a

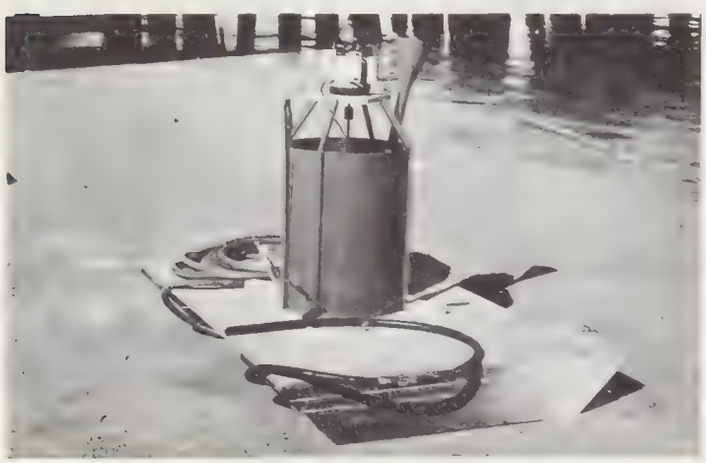

FIGURE 5. — Tank for mixing tracer and sediment.

mixing propeller, located at the bottom of the tank. A bottle-crusher tube, which terminated just above the propeller, was perforated to allow the slurry to circulate through the lower end of the tube. A hydraulic ejector was located outside the tank with the suction line entering from the top of the tank and extending to the bottom along the side. The discharge line consisted of a hose weighted at the end to place the sediment mixture on the bottom.

Instrumentation

Instrumentation requirements for tracing operations are rather complex and necessitate the use of a boat equipped with a stable alternating-current generator. The system consists of a sled-mounted radiation detector (fig. 6)

FIGURE 6. — Radiation detector mounted on sled.

that is towed along the bottom behind the survey boat. The detector is connected by a coaxial cable to the electronic equipment located on the boat.

The detector uses a scintillation crystal (thalium-activated sodium iodide) mounted in an aluminum cylinder and coupled to a photomultiplier tube with a three-stage transistorized preamplifier. The light pulses generated in the scintillator are converted to electrical pulses by the photomultiplier tube, and the preamplifier then increases the pulse power for transmission through the cable to an amplifier-analyzer located on the boat. The scintillation detector is more sensitive than a Geiger-Muller (G-M) system, and pulses produced by the scintillation detectors are proportional in size to radiation energy, whereas those from a G-M system are not. The pulses are amplified and fed to a discriminator circuit that counts or rejects the pulses, depending on the pulse height. Since the pulse height is proportional to the energy of the incident particle, energies above or below the desired level are rejected. This feature is desirable in that naturally occurring radioactivity or fallout does not interfere with the tracer activity measurements. As the energy of radiation from most isotopes is spectral, a band of energies is usually observed in which the greater portion of incident particles or rays from the isotope in question can be counted.

A strip chart recorder records continuously the pulse rate (which is proportional to bottom activity) in the area being covered. These recordings are marked at time intervals that can be correlated with location for plotting.

Survey Procedures

The survey boat at any time was located through the use of either sextants or surveyor's transits. Both methods were used with position fixes being made at predetermined time intervals or called intervals. Simultaneously with position fixes, time marks were placed on the activity recording so that the correlation could be made. Direct interpolation was made between location points on the charts for plotting variations in activity. Of the two systems used, the sextant method was more effective, since the observers were on the boat and plotting of survey paths was done as the operation proceeded. This system of constant location lends itself to better coverage of the affected area, as partially surveyed areas can be covered to define completely limits of material deposition.

Since the accuracy of the information obtained is no better than the accuracy of location measurements, constant checks and rechecks of location can be made by crossing and recrossing courses already run and visual checks between objects located on the charts can be made. The largest scale maps available were used for navigation and data reduction to reduce possible error.

The background radiation survey before deposition of the radioactive material is very essential, not only because it establishes a datum plane for later observations but also because the mathematical significance of activity measurements is a function of the background levels. For these reasons one must be certain that the area is thoroughly surveyed and that all areas of high or low activity are completely defined. Since silts, clays, and organic materials possess more natural radioactivity than sands or gravels, it is possible to construct (through careful data analysis) a deposition chart that defines the various material locations including hard clay areas. By drawing activity level contours, a marked resemblance can be seen of soundings showing deposited material. Thus background surveys can be used as guides to probable deposition areas for the after-placement surveys.

Placement of labeled sediment with optimum tide or flow conditions to obtain maximum movement is a situation that is difficult to obtain. In tests conducted by the Waterways Experiment Station, the assumed extreme conditions have been used as criteria for times of placement. At Galveston Bay, for example, the inshore deposits of material along the North Jetty were made at low slack water whereas the outer deposits were made at high slack water. This was done to prevent movement of the tracer while the material was in suspension. The times of placement of material in the Cape Fear River, however, were selected to encompass both extremes; that is, deposits were made with rising and falling tide conditions. This was done because the bulk of the sediment would remain in suspension for a sufficient time to allow movement regardless of the time of deposit.

The postplacement surveys were conducted in the following manner: After the material was placed, the survey operation was undertaken immediately. No set pattern of coverage or route was established. The areas of high activity were defined by sufficient tracing. Problem areas or areas of special interest were covered as time permitted. In the case of slow movement of the tracer, daily surveys reveal development patterns or gross tracer movement. Severe weather conditions or rapid movement in suspension will leave no evidence of a "trail" or indicated direction of movement. In these cases general coverage of the area will show pockets of material or sometimes random distribution of particles over large areas. After dispersion has progressed to the point where activity falls below the significant levels or the development patterns begin to recede, surveying may be discontinued.

Safety precautions as outlined in the license application were followed during the entire surveying operations. In particular, all beaches and shores in the immediate vicinity were monitored to insure no contamination. The boat (from which all operations were conducted) and the placement equipment were checked to insure freedom from harmful radiation. Significantly, the largest dose rate observed during the Galveston test came from the luminous dial of the boat's clock.

Data and Results

Background surveys were plotted in the form of contour maps. Each daily postplacement survey was reduced in a similar manner, the background subtracted to yield net count rate, and this value corrected for the decay of the radioisotope. The decay process of any radioisotope is an exponential function that follows the general equation $I=I_o e^{-\lambda t}$ where I is the present intensity of radiation, I_o is the initial intensity, e is the base of natural logarithms, λ is the decay constant for a particular isotope, and t is the elapsed time. For gold ^{198} (half-life 2.7 days) the activity correction factor for four elapsed days would be 2.8 by which those significant net readings would be multiplied. After the decay factor was applied to the count rates, these values were plotted on the charts in the proper locations. At this point, either activity contours

may be drawn to show daily movement patterns or areas of significant deposit may be shown also by contours or by cross-hatching. In cases where sufficient coverage was not obtained to define boundaries of the area affected, points indicating activity level were plotted on the survey path to show that activity was observed.

The initial observations made at Galveston showed general movement inshore from the inshore locations, with some rapid movement from severe weather encountered during the survey operation. Activity contours showed the dispersal of material with a small fraction of material passing through a small boat pass in the jetty (fig. 7). Since results did not show conclusively whether the bottom material went through or around the jetty into the ship channel, six additional deposits were made in deeper water near the end of the jetty. These tests showed considerable movement of tracer material around the end of the jetty, with random depositions in the ship channel and in the area between the two jetties. Movement of material was dependent upon weather conditions in the area. That is, it appeared that wave action caused the tracer material to be in a semi-suspended state wherein it was moved readily by the current. In one test, tracer material was observed 5,500 ft. from the drop location within 4 hours after placement. When waves were small, movement was slow. For long-term operations at Galveston, another license for using an isotope of long half-life (such as scandium [46] with a half-life of 85 days) would have been necessary. This, however, would permit determinations of transport processes and allow more comprehensive studies to be made.

Observations made in Cape Fear River showed deposition of material in the problem area within a short time regardless of the tide conditions at the time of placement. In fact, movement was so rapid that delineation of the actual paths of tracer material was not possible. However, the purpose of the test was served in that tracer material deposited in the area was detected in the wharf basins (fig. 8). Hence, it can be concluded that the dredged spoil area should be diked. One test, made with tracer

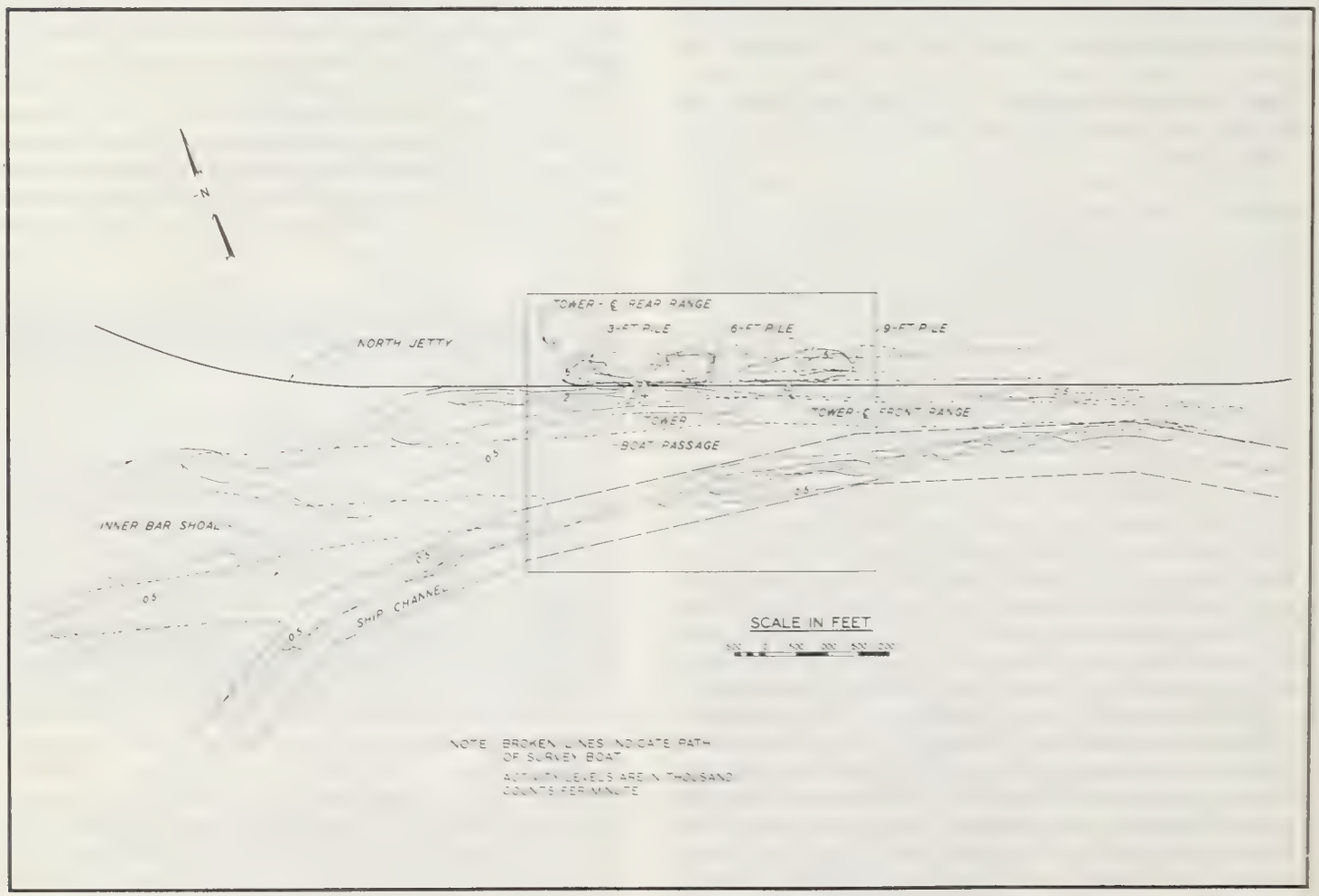

FIGURE 7. — Radioactivity contours on 10 June 1961; Galveston tracer tests.

material deposited on a shoaled area in the river well upstream from Sunny Point Army Terminal, revealed considerable shoaling of the dredged area by material transported by the river. Therefore, the shoaling problem at Sunny Point may only partially be alleviated by diking the spoil area.

Closing Remarks

The use of radioactive material for tracing sediments is only one tool to be employed in the overall analysis of a particular problem. It has limitations, but under certain situations can provide worthwhile results. Generally the information obtained from tracer tests is of a qualitative nature and pertains to relative rates of shoaling or concentrations of material in certain areas. Quantitative measurements are extremely difficult at present because of the nature of sediment movement and the tracer material itself. For example, as the detector passes over the bed there is no method of determining the difference between a single large particle of tracer or several small particles. Neither is it possible with the adsorption method of labeling to determine the exact amount of label that is adsorbed to the sediment particles. If any of the labeled sediment is shielded by being covered with inert bottom material or moves in suspension above the face of the detector, the observed counting rate is decreased. The inherent safety hazards resulting from use of long-lived isotopes prevent tests of long duration.

In spite of limitations, tracing techniques offer the only method for continuously tracing sediments in deep water without direct sampling. In addition, large areas can be studied within a relatively short time. Further study and development of techniques and equipment may reduce or eliminate many of the limitations currently faced.

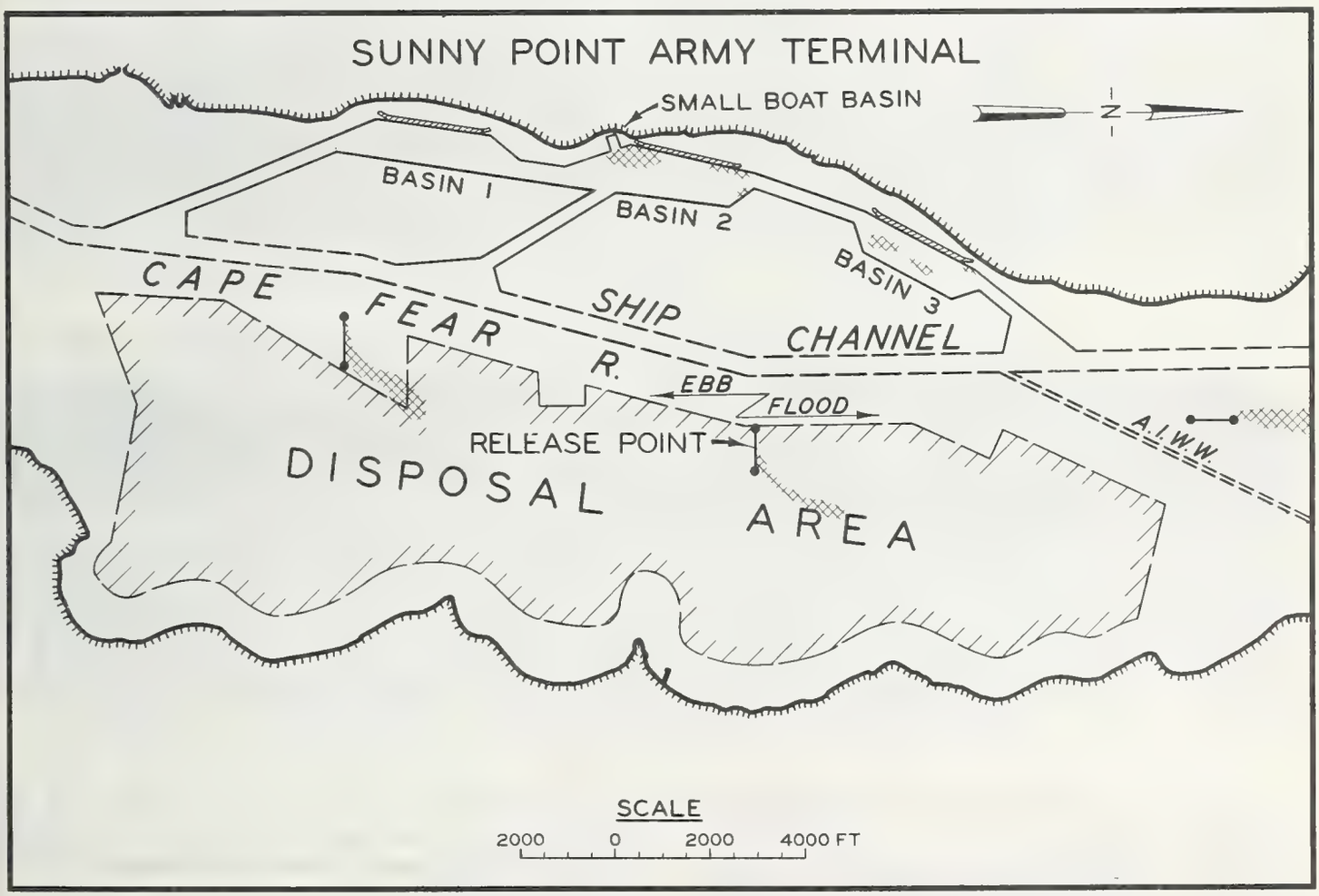

FIGURE 8. — Sunny Point Army Terminal tracer release points and deposition areas.

USE OF THE SWEDISH FOIL SAMPLER FOR TAKING UNDISTURBED CORES OF RIVER BOTTOM SEDIMENTS[1]

[Paper No. 63]

By R. J. PICKERING, *geologist, Water Resources Division, U.S. Geological Survey, Oak Ridge, Tenn.*

The release during the past 20 years of very small but measurable amounts of radionuclides into the Clinch River in eastern Tennessee by the Oak Ridge National Laboratory has created a unique opportunity for determining the distribution and behavior of fission products in a natural stream. A joint study of the fate of radionuclides released to the Clinch River has been carried on by the Water Resources Division of the U.S. Geological Survey and Oak Ridge National Laboratory since February, 1960, as part of the general program of the Clinch River Study Steering Committee.[2] Undisturbed core samples of river bottom sediments were required for phases of the joint study pertaining to the vertical distribution of fission products in the sediments.

Preliminary coring tests with several sampling tools showed that the relatively simple tube-type samplers were unsatisfactory for use in the Clinch River. With such devices there was preferential coring of certain sediment layers and excessive compaction of the sediments in the core sample. Driving of thin-walled tubes having inside diameters of 3 to 4 inches into the river sediments caused less compaction of samples than the driving of tubes with diameters of 1 to 2 inches,[3] but better control of compaction was necessary.

Further tests and inquiry resulted in the selection of the Swedish Foil Sampler, which takes cores that are slightly less than 3 inches in diameter, for sampling bottom sediments in the Clinch River. The Foil Sampler is a piston-type sampler in which thin axial metal strips (foils) are used to decrease friction between the sediment core and the sample tube while coring is proceeding. Cores as much as 14 feet in length were obtained in water as much as 45 feet deep with the Foil Sampler mounted on a barrel float equipped with a drilling tower. More than 100 undisturbed cores were collected at 14 cross sections on the Clinch River and at four cross sections on two tributary streams.

The loading of the sampler is illustrated in the series of photographs shown in figures 1 to 4. Steel "foils," fed from coils in the sampler magazine, are attached to the piston assembly (fig. 1). The piston assembly is then pushed up into the sample tube a short distance, pulling the foils down around the lip of the tube and up along its interior surface (fig. 2). After the

FIGURE 1. — Attachment of foils to piston assembly, Swedish Foil Sampler, showing sample magazine, foil fasteners, and piston guide.

magazine cover, cutting shoe housing, and cutting shoe (fig. 3) have been attached and the piston guide has been locked into the base of the piston assembly, the Foil Sampler is ready for lowering (fig. 4).

The distance from the water surface to the sediment surface must be measured accurately with a weighted line before coring can proceed. The sampler is then lowered to within a few inches of the sediment surface, and the piston

[1] Work done at the Oak Ridge National Laboratory, operated by Union Carbide Corp. for the U.S. Atomic Energy Commission.

[2] The Clinch River Study Steering Committee is composed of representatives of the Oak Ridge National Laboratory and six State and Federal agencies that have an interest in the Clinch-Tennessee River system. It is responsible for the investigation of the distribution, behavior, and effects of radioactive materials in the river system.

[3] P. H. Carrigan, personal communication.

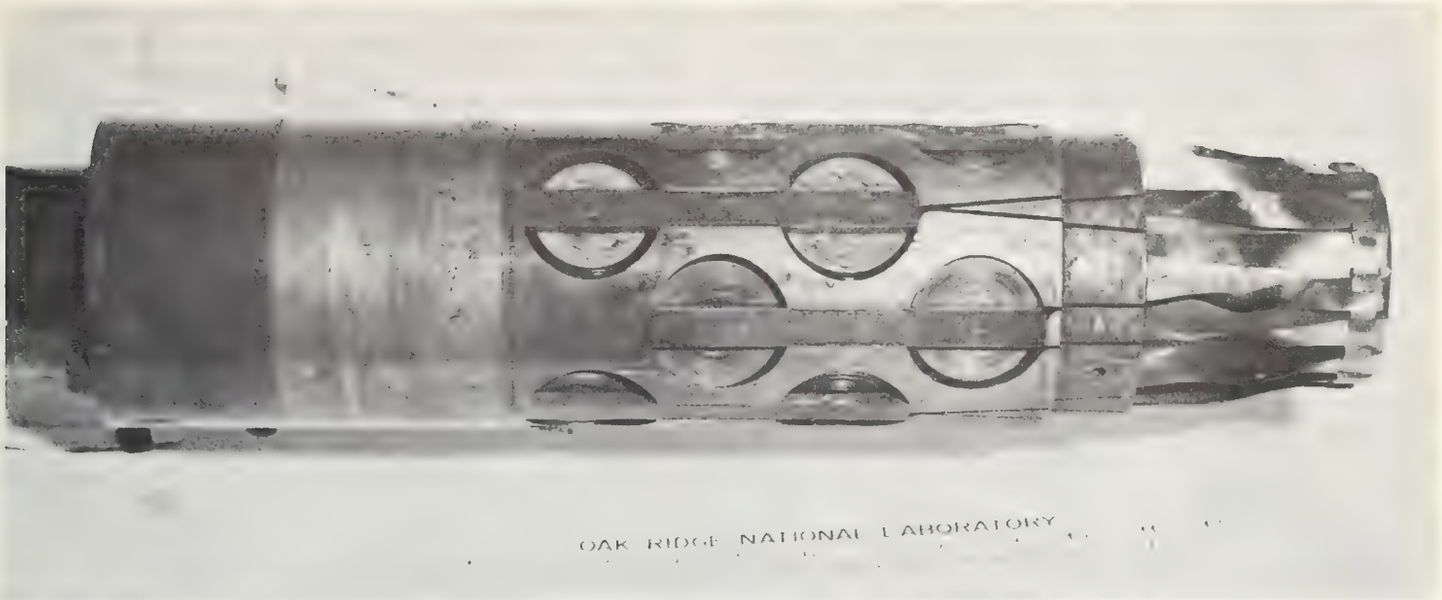

FIGURE 2. — Close-up of sampler magazine, Swedish Foil Sampler.

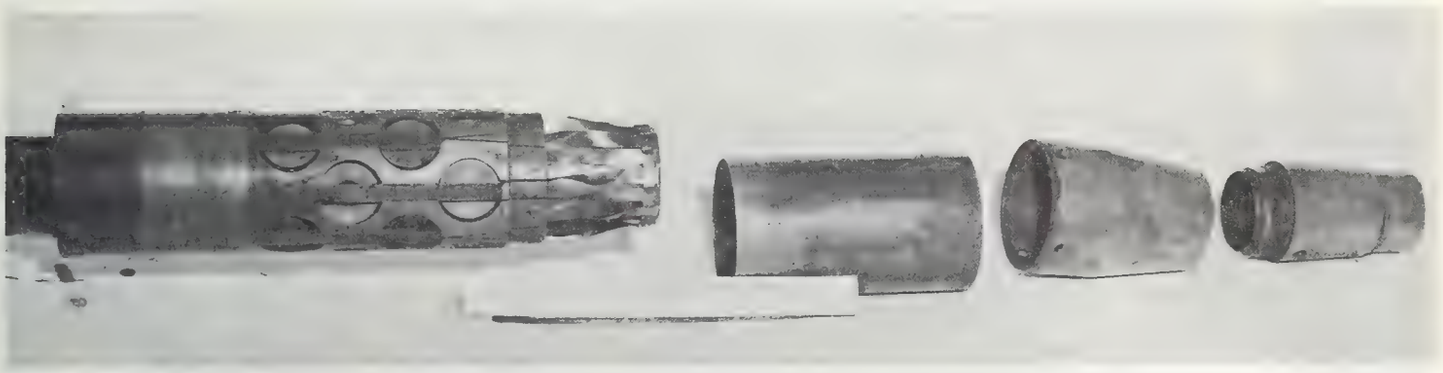

FIGURE 3. — Exploded view of sampler head, Swedish Foil Sampler. Left to right: sampler magazine, magazine cover, cutting shoe housing, cutting shoe.

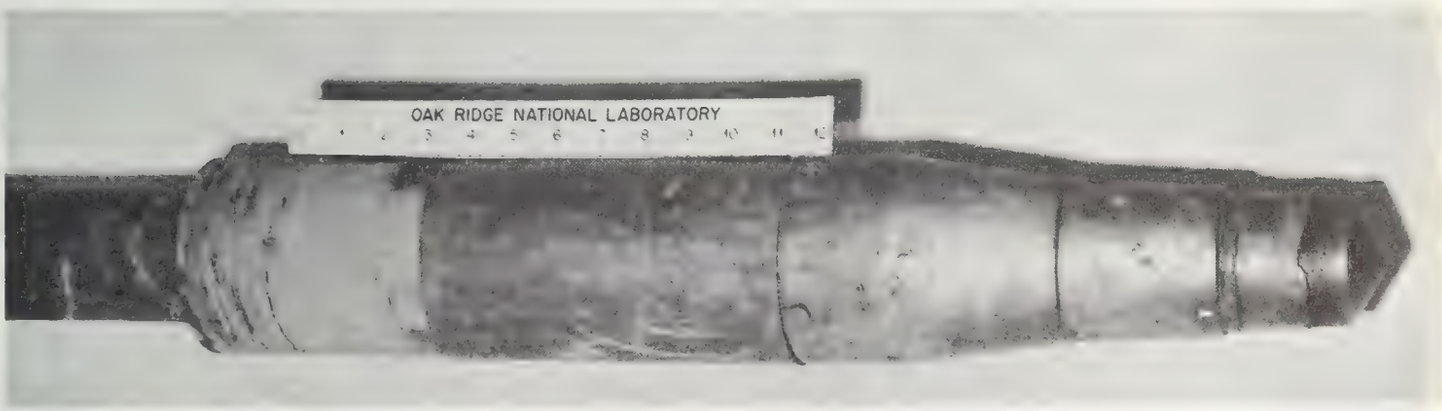

FIGURE 4. — Sampler head ready for sampling run, Swedish Foil Sampler.

chain is fixed to a hook at the top of the drilling tower. As the sampler is pushed into the sediment, the foils unroll and form a sheath that encloses the core and prevents its contact with the interior of the sample tube. The core does not move relative to the foil strips, and the only friction in the sample tube is the metal-to-metal friction between the foils and the interior of the tube. Compaction and blocking of the core in the tube due to friction between the sediment and the tube is thus prevented. During the insertion of the sample tube into the sediment, the piston and the core of sediment do not move from their original positions.

The primary function of the piston in a piston sampler is to prevent loss of core while the sampler is being raised to the surface. Because of the rigid sample tube and the tight-fitting piston immediately above the core, the hydrostatic force due to the column of water above the base of the sample tube can act on the core only in an upward direction, thus holding it in the tube. If the core begins to slide out of the tube, a partial vacuum is created between the top of the core and the piston, which is locked in position while the sample tube is being raised, thus increasing the differential upward push of hydrostatic pressure on the core and inhibiting its further movement. Adhesion of the core to the metal foils of the Swedish Foil Sampler also tends to hold the core in the tube.

Cores collected from the Clinch River were transferred from the metal sample tube of the Foil Sampler to a slightly larger plastic storage tube in the manner illustrated in figure 5. Because the plastic tube could be slipped over the sample tube, it was possible to transfer the core, still encased in its foils, without sliding it through the entire length of plastic tube. After the cores were transferred, they were refrigerated in order to suppress biological and chemical action.

Under certain sampling conditions, satisfactory cores could not be obtained with the unmodified sampler. Recovery of river sediment was excellent at points near the bank where the river sediment overlies soil, but in the deeper parts of the river channel where soft sediment lies directly on bedrock, the lower 6 to 18 inches of the sample were commonly lost. At some sampling sites, the driller was able to minimize core loss by disconnecting the piston chain while the sample tube was still in the sediment and then raising the sampler as quickly and as gently as possible, but this procedure was not always adequate. To decrease further core loss, a sampling shoe containing a basket-type core retainer was designed for the sampler and, upon request, was fabricated by Sprague and Henwood, Inc., the drilling contractor. The

FIGURE 5. — Transferring core from sample tube to plastic storage tube.

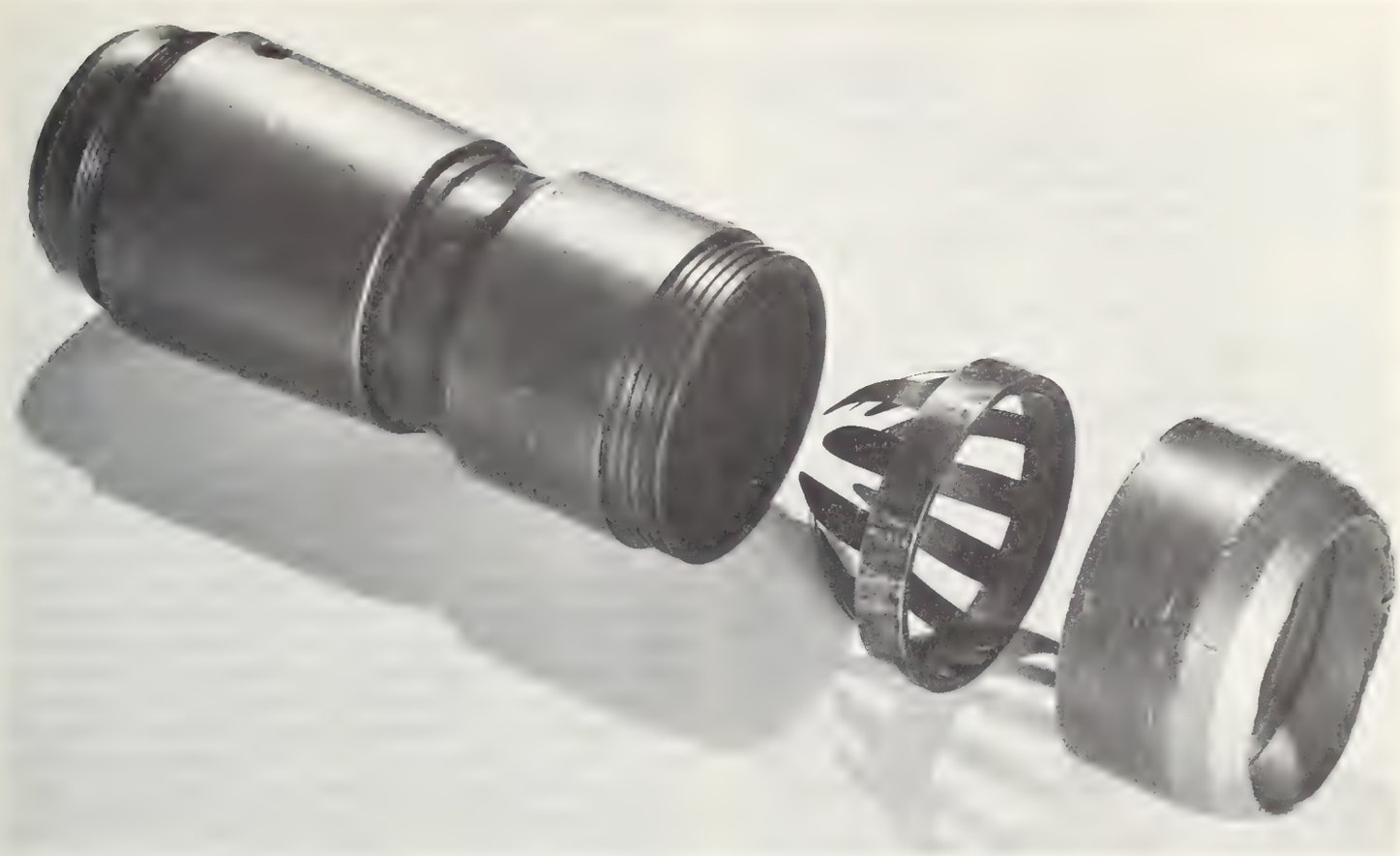

FIGURE 6. — Basket shoe assembly, Swedish Foil Sampler.

"basket," which consisted of curved, closely spaced, flexible spring steel fingers attached to a steel ring, was inserted near the base of the cutting shoe (fig. 6). Above the basket was fixed a thin plastic sleeve that was slit at the top so it would collapse over the basket when the sediment began to slide out of the shoe and thus partially, or at times completely, seal the opening at the base of the sample tube. Table 1

TABLE 1.—*Bottom sediment coring results, Clinch River, at mile 10.0*

Hole No.	Depth of water	Sediment penetrated	Percentage of core recovered	
			Without basket shoe	With basket shoe
	Ft. In.	*Ft. In.*		
1	8 9	6 9	75	.....
2	19 8	8 7	90	.....
3	22 5	3 8	59	.....
3	23 2	3 8	.....	80
4	16 6	8 4	96	.....
5	7 5	4 6	78	.....
6	8 7	2 4	0	.....
6	9 3	2 0	.....	92
7	8 4	8 9	79	.....
8	17 8	6 3	92	.....
9	27 9	2 9	0	.....
9	28 10	4 1	.....	78
10	4 9	5 9	93	.....

provides an example of the excellent results that were obtained with the basket shoe. One hundred percent recovery of a 10-inch-long core, composed entirely of sediment with a consistency like that of thick gravy, was obtained at one sampling site. The nearly watertight seal formed by the collapse of the plastic sleeve over the basket made such excellent recovery possible. The basket shown in figure 6, which has more widely spaced fingers made of heavier spring steel than the basket used in the fine sediment, was designed for recovering sediments that have a high sand content.

Particle-size distribution in the sediment is fairly constant throughout the portion of the Clinch River that contains radioactive sediment. The sediment is composed of approximately 15 to 20 percent clay, 25 to 30 percent sand, and 50 to 60 percent silt.

Core recovery at sampling points where radioactivity is believed to be present throughout the full thickness of the river sediment generally ranged from 80 to 100 percent (see table 1). Poor core recovery was accepted only where the sediment consists of nearly pure sand or of a mat of leaves and twigs overlying soft mud, but few such localities were encountered.

DISCUSSION — SYMPOSIUM 2

Sediment in Streams

Moderator: Thomas Maddock, Jr., USGS

Panel: H. A. Einstein, University of California
V. A. Vanoni, University of California
E. L. Pemberton, BR

MR. MADDOCK:

I've selected a panel to help on this problem and I think I have a real good one. The first one is Dr. Hans Albert Einstein, who, we might say, is Mr. Sediment Movement himself. I don't think anybody can work with the movement of sediment or solve any sediment problem without having been exposed to his work. The next one is Dr. Vito A. Vanoni, who I think is equally well known all over the country. Certainly my own association with him has extended over 26 or 27 years. We also have one of the younger men in the sediment field, Ernest Pemberton, who is assistant head of the Sedimentation Section in the Bureau of Reclamation in Denver.

The question I want to ask the panelists to discuss is this: The single thread holding together our model studies and our work in the field in trying to design stable channels, whatever it may be, is the relation between roughness and sediment load. We find that to get a stable channel these two things have to be compatible in some way. To start out the discussion on this question, I would like to ask each panelist, beginning with Dr. Einstein, to spend some time trying to bring out some of the elements that are to be found in this kind of a problem and what we have to do about them.

DR. EINSTEIN:

I hope I understand the question properly as to mean that we ought to answer whether both the friction factor and the sediment transport are related or unrelated. Is that about the meaning? In the design of any kind of a channel that carries sediment, there is such a tremendous combination and variety of various cases where once one of the variables and once another one of the variables is given and some of them are unknown, that it is almost impossible, in general, to answer this question. What I would like to say, and what I have been saying all along, was that there exists at least two relationships that govern all these flows and these are two simultaneous relationships such that, given a whole set of conditions, two of the variables can be determined and predicted for each case. Now these two may be the friction and the sediment load or, in other words, what I'm trying to say is that if all the other variables are given, then these two can be predicted and for that reason are naturally interrelated. I would say that the two are definitely interrelated in the sense the question was asked.

I would like to make one more remark and that is, looking around this room, I am somewhat reminded of the last meeting in Denver of this corresponding group, and I wouldn't want the moment to pass without having an occasion to express my satisfaction that today the design and field engineer contributes at least as much to the knowledge in our field as the laboratory does. At that time field and design engineers came to the meeting in order to learn what the relationships are, what they could calculate that they had been guessing at before. Today we're being taught which of our doings are usable and which are not. I think this progress is tremendous and I would like to congratulate you for that and let's give everybody who is involved in the field work and has contributed so much a good hand right now.

MR. MADDOCK:

Thank you very much Dr. Einstein. I would like to ask Dr. Vanoni the same question. I think from some of the discussions we've had that he may express this a little bit differently. This is a matter of semantics, and I think that if you make a question broad enough everybody will interpret it differently, and you will get a different answer. I hope this is the case as I ask Dr. Vanoni to talk about the same thing.

DR. VANONI:

Thank you, Tom. The question of the interrelations between the friction factor of an alluvial stream and the transport rate of sediment is one that has been discussed at length in the literature during the last 15 years. I think it is clear from laboratory studies that there is a relationship. For instance, we know that when a bed is covered with dunes, it is hydraulically rather rough, and the velocity and sediment transport rate over it tend to be relatively small. Now if we increase the velocity by increasing the discharge or slope, the dunes will tend to be modified so as to reduce the friction factor and increase the sediment transport rate. This is something we've learned from the laboratory.

Some river engineers knew about this relation quite a few years ago. I recall the work of Buckley, on the Nile River in 1922, in which he

plotted river stage, hydraulic radius, mean velocity, friction factor, and suspended load discharge against time for a whole year. It was clear from this that when the suspended load discharge increased, the water discharge also increased and that there was a tendency for the friction factor to decrease. This relation was rediscovered by many of us, including myself. It came as a discovery when we should have known it to start with.

The problem of predicting the friction factor is very closely linked with that of predicting the bed form. We know from laboratory studies that the presence of suspended load tends to decrease the friction factor, but we also know from laboratory experience that this is not the major effect in reducing the friction factor. The big reduction in friction factor results from changing the size and shape of dunes. Now I realize that a flume is not an accurate model of a river but, just the same, I think it is clear from observations that the same kind of changes in roughness occur in rivers as in flumes. So our problem is to predict for what conditions the dunes are going to start to wash away because this is when the biggest change in roughness is going to occur.

You have seen Dr. Simons wrestle with this problem this morning for a rather simple case of a flume with one material, and you can see that he got into some pretty complicated mathematics and relationships. I'm afraid we're not ready yet to predict the roughness relation on a river, because we really can't predict it in a flume. Here is where one of our big problems lies in predicting sediment load of a stream. One might say that this is the problem of working out the hydraulics of alluvial streams.

The trend in recent years to consider the stream as a whole rather than only the sediment discharge represents a move in the right direction. For several years many of us focused our attention on the suspended load or bedload and ignored the hydraulic factors of velocity and friction. I'm sure everybody didn't do this but many of us did. I think we are more realistic now that we are looking at the stream as a whole.

MR. MADDOCK:

Thanks very much, Vito. I'm going to call on Ernie Pemberton. Mr. Pemberton is one of those of many among us who has the problem of prediction, because that's what design is. You make a certain set of assumptions, and then you predict what's going to happen and you hope the verification confirms your decision. Now, inasmuch as Dr. Vanoni has indicated there may be some question as to whether you can do this or not, I want Mr. Pemberton to explain how he does it and some of the problems that he has encountered.

MR. PEMBERTON:

Thank you, Tom. I think from the question and the response today that there still remain many problems in predicting the sediment load of a stream. I consider it a privilege to be one of this distinguished group and will admit that I've had my feet wet with regards to the collection of field data.

In the collection of data, we have tried to analyze some of the variables that take place in sediment transport and to obtain a better knowledge of the existing conditions of rivers. I'd like to bring up one additional point on the same subject to which Dr. Einstein and Dr. Vanoni responded. That is, the bank friction factor as related to sediment transport. We have encountered many problems in the Bureau of Reclamation on sediment transport in canals or in rivers which involve channelization works where bank friction is an important factor. Dr. Einstein pointed this out to us a number of years ago, and I still don't believe we know how to handle the bank friction problem as related to sediment transport.

I would also like to comment on the application of material presented by Dr. Simons and Dr. Kennedy on sediment transport. We in the Bureau of Reclamation are continually working with problems of river channelization for channels that are aggrading or degrading. This involves trying to estimate the total sediment load in aggradation or degradation areas and requires a better knowledge of sediment transport in order to do something about the problem. Also, we need to know the total sediment transport of channels above reservoirs and diversion dams. At the diversion dam, we want to know how much of this sediment will be diverted and carried by the canal. I think the papers presented have given us more useful tools to aid in these problems. I'm not sure whether the solution to some of the problems as presented are applicable to other rivers. As an example, I'm sure many of you are aware of the problems involved in taking the Missouri River data and using them on a little stream out in Nebraska which was very dissimilar with regards to friction factors, the suspended sediment transport and total sediment transport. I hope this has answered the question.

MR. MADDOCK:

Well, frankly, I think you had an impossible question to start with, but I was wondering what you did about it and we did find out something. We found out something about what you were trying to do about it. We have about three more minutes before this meeting is supposed

to be devoted to the announcements. There were a large number of questions that came in that had to do with the details of the papers. I think that each and every one of us had questions about some part of some paper. I know I have questions about almost every one of them. Some things I disagreed with and on other things I would like additional explanation. It just doesn't seem possible that we could really accomplish very much if we took up these individual problems one at a time.

Pemberton brought up one just a moment ago, and that is how well can you apply what you have learned in the model to a natural stream. We can sit around and debate that one all afternoon. There are a number of the wonderful debatable problems and discussible problems in this particular field. Unless somebody has a question that he would like any of the people on this panel to answer (and I think that this is a real opportunity for asking questions you might not have another opportunity to get), I turn the program back to Don Bondurant.

Symposium 3.—Sedimentation in Estuaries, Harbors, and Coastal Areas

INTRODUCTION

Most papers presented in this and the 1947 sedimentation conference were concerned primarily with the erosion, transportation, and deposition of sediment in streams and reservoirs. Of equal importance, particularly in coastal areas, is the behavior of stream-borne sediments upon entering the ocean. Such sediments may enter directly on an open coast or they may enter into an estuary. The practical problems involved in these two processes differ greatly and are discussed briefly as follows.

A stream discharging directly on to an open coast will drop its sediment load upon entering the relatively quiet water of the ocean. The bed materials load will be deposited in the form of a delta, whereas the wash load will be carried offshore into deep water either as a density current or in suspension by the rather feeble coastal currents. Whether or not the delta composed of the coarser sediment fractions exists as a pronounced bulge on the coastline depends on the size of the sediment, the range of tide, but principally on the intensity of wave attack. If the wave action is relatively light and the material is relatively coarse, then the delta may be of a rather permanent form. On the other hand, if the intensity of wave attack is high and the material fine, then the stream-borne sediments are resorted by the high turbulence of the breaking waves and moved along the coast by the littoral currents, and no evidence of a delta in the usual sense exists.

In many localities along the coastlines of the world, streams often enter a bay or estuary before the river waters reach the ocean. The amount of sediments transported by such streams that may reach the ocean depends on a number of factors, such as stream discharge, size of the bay or estuary, and character of the sediments. In case of large bays or estuaries and moderately sized streams, almost all the bed material load will be deposited in the bay and only some of the washload is eventually transported to the ocean by streamflow and tidal currents. A great percentage of the washload actually may be deposited in the bay or estuary as a result of flocculation, which occurs when the sediment-laden fresh water meets the saline bay waters.

Although streams appear to be the major source of sediments supplied to our coastlines, such factors as cliff erosion, slides, onshore movement by wave action, and wind action may be of importance in many regions. In addition to the supply of sand, the problem of the loss of sand may also be important. Such losses may result from the movement of sediment offshore into deep water, losses into submarine canyons, accretion against littoral barriers, removal of sand for construction purposes, wind action, and abrasion by wave action. While all of these factors are not discussed in this morning's session, it is gratifying to see the amount of time that has been alloted to this subject.

IMPORTANCE OF MARINE INFLUENCES IN ESTUARINE SEDIMENTATION

[Paper No. 64]

By Robert E. Burns, *research oceanographer, Office of Research and Development, U.S. Coast and Geodetic Survey* [1]

The important early settlements in this country were in the estuaries of major rivers. Consequently, it is not surprising that many investigations of estuarine sedimentation have been made in these same estuaries. Under these conditions, it is important to realize that most contemporary concepts of estuarine sedimentation have been strongly influenced by the pres-

[1] Field work and analysis of sediment in Port San Juan, British Columbia, was conducted at the University of Washington under Office of Naval Research Contract Nonr-477(10), Project NR 083 012.

ence of an overwhelming dominant river load. Although initially there was a tendency to consider the estuary merely as a continuation of the river environment, it was recognized early that marine processes must also be considered. Since the estuary is an environment in which the marine forces are at least in equilibrium with the river forces (as contrasted with, for example, the deltaic environment), the importance of these marine forces should be considered at some length.

Although the role of tides and tidal currents has been recognized for some time, only in recent years has the importance of salinity and the saline-wedge intrusions been examined in detail. The role of the salinity distribution is now the subject of many investigations, and in general terms its effect on both the net-circulation and on flocculation of suspended clay mineral load is reasonably well understood.

Unfortunately, because so much of the research has been conducted in estuaries of major rivers, examination of the behavior of river-introduced load has tended to overshadow examination of other sources of sediment and of the nonriver processes. The saline wedge is important in any estuary, but its impact upon sedimentation becomes relatively less important as the magnitude of the suspended clay mineral load of the river decreases.

In many nonmajor river estuaries, the problem of evaluation of sedimentation is extremely complex because of the secondary importance of the river-related source of sediment and the absence of a major river flow. In these areas, an understanding of the various sources of sediment other than a major river and an evaluation of the importance of the processes of nonriver origin are necessary to understand the overall sedimentation regime.

One of the simplest possible models for sedimentation is that in which the characteristics of the sediment are determined by the interaction between those processes tending to remove sediment and those processes tending to add sediment. If the sediment is characterized as a function of the mineralogy, diameter, shape, and lattice of the component grains, this interaction can be expressed in the form:

$$S_o = (S_i + S_s) - S_r$$

Where S = A function (mineralogy, shape, diameter, lattice);
$_o$ = Observed at time of survey;
$_i$ = Initial condition (presurvey);
$_s$ = Added from source;
$_r$ = Removed from area.

Despite its simplicity, this relation is the basis for more elaborate models, and in itself is of value in determining changes in sediment over periods of time. Although the model is rarely expressed in these specific terms, it has been used by many investigators of estuarine sedimentation. Since adequate evaluation of even this basic model is generally beyond the capabilities of the investigator, attempts to examine natural situations usually involve one of two basic approaches.

(1) The more common approach assumes a steady state in which the observed sediment type is unchanged through time, i.e., $S_o = S_i$ and S_r must equal S_s. The sediment found within the estuary is assumed to be in equilibrium with environmental processes, and an attempt is made to determine the physical processes controlling this equilibrium.

Geologists examining lithified samples from the geologic column are invariably limited to these steady-state assumptions by the nature of their data. Further, many studies of contemporary conditions follow this line of reasoning because of the extreme difficulties in conducting more adequate sampling programs.

(2) The second general use of the basic model involves a system in which $S_o \neq S_i$. Evaluation of this assumption involves at least two surveys of the area and an attempt to determine representative values of both S_s and S_r for the period between the surveys. A comparison of charts based on past surveys with more recent data is frequently used to evaluate S_s and S_r.

However, in this simple form the model gives no insight into the various processes acting within the area of investigation nor is it useful in determining the particular processes responsible for additions and removals of sediment. A further development of the model is necessary in order to consider the behavior of sediment within the estuary and to evaluate the various processes responsible for sediment distribution.

One of the simpler models of estuarine sedimentation that can be developed is presented in figure 1. This indicates that all sediment introduced into an estuary, from whatever source, has one of five possible behaviors:

(1) The sediment can move through and out of the estuary without ever being deposited (a process of bypassing).

(2) The sediment can move through and out of the estuary in a series of steps involving alternate deposition and erosion.

(3) The sediment can be permanently deposited after initial deposition.

(4) The sediment can be subjected to alternating deposition and erosion through one or more cycles and can then be permanently deposited.

(5) The sediment can be recycled continually without ever being moved out of the estuary or being permanently deposited.

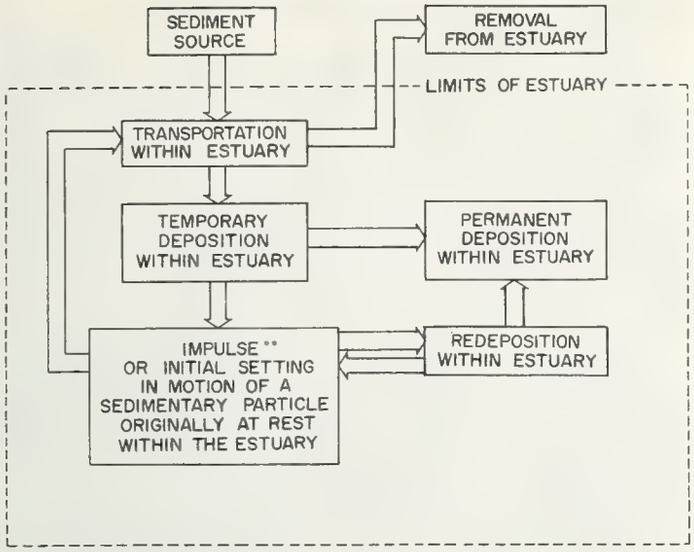
SEDIMENT SOURCE
REMOVAL FROM ESTUARY
LIMITS OF ESTUARY
TRANSPORTATION WITHIN ESTUARY
TEMPORARY DEPOSITION WITHIN ESTUARY
PERMANENT DEPOSITION WITHIN ESTUARY
IMPULSE ** OR INITIAL SETTING IN MOTION OF A SEDIMENTARY PARTICLE ORIGINALLY AT REST WITHIN THE ESTUARY
REDEPOSITION WITHIN ESTUARY

FIGURE 1. — Block model for evaluating sedimentation in an estuary.

Each of these five behaviors may vary in importance through time, and all of them may be operating simultaneously but on different sedimentary particles. The observed bottom sediment distribution is, then, the result of the summation through time of all the processes acting in the area.

The application of this model to a natural situation involves the evaluation of the various sources of sediment and the processes of transportation and erosion within the area. In particular, we must consider the processes that may be capable of giving impulse to particles which for one reason or another are at rest on the bottom.

Figure 2 is a generalized presentation of the various sources from which sediment may be introduced into an estuary.

The surface source is responsible for the

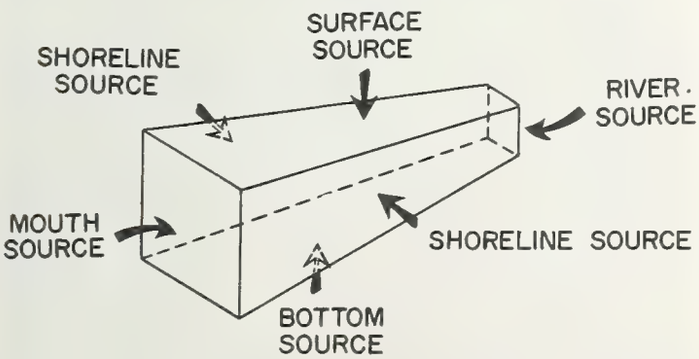

FIGURE 2. — Generalized sources of sediment entering an estuary.

introduction of such material as volcanic ash, rafted sediment, windblown particles, etc. Although some specific incidents of importance have been reported (*11*), such sources are usually minor and are not generally considered in the evaluation of most natural situations.

The bottom of the estuary may be a source of sediment introduced into an estuary by processes related to turbidity currents. Although such processes are sometimes important in redistributing sediment already within the estuary, it is unlikely that the bottom is normally an important source of new sediment in estuaries.

Without a river source the estuary cannot exist, but the importance of the sediment added from this source both varies greatly with time in any single estuary and certainly from one estuary to another. In the estuaries of large rivers the river is usually a dominant source of sediment, and several investigators (*2*, *4*) have emphasized processes that distribute the river-introduced sediments. When the river is a dominant source — specifically when the river is a large one with an extensive drainage basin — the problem of flocculation of silt- and clay-sized particles is of major importance. However, in areas of limited drainage basins and relatively little weathering to form clay minerals, the problem of flocculation becomes considerably less important.

An additional factor affecting the sedimentary-laying importance of the river source is the general circulation related to the intrusion of the saline water along the bottom. This has been covered in detail from the hydrodynamic viewpoint by various investigators (*9*). The effectiveness of this circulation in trapping river-introduced sediment was discussed by Schultz and Simmons (*12*). When the river flow is diminished, as in the case of many of the nonmajor river estuaries, downstream movement is confined to the upper layers or may be completely overbalanced by the tidal currents. In these cases, all but the very finest sediment is trapped within the limits of the estuary by some initial deposition. In keeping with the model (fig. 1), further resuspension and transport must then be related to processes that are not directly attributable to river action.

The shoreline source exists in the nearshore water within the estuary, and it contributes sediment from along the beaches and shoreline, including that produced by slumping of the shore. The importance of sediment from shoreline sources has been minimized in many of our investigations of major river estuaries, but it is considered of primary importance in the north European estuaries (*3*) and in the Puget Sound area.[2]

[2] WANG, F. H. RECENT SEDIMENTS IN PUGET SOUND AND PORTIONS OF WASHINGTON SOUND AND LAKE WASHINGTON. 160 pp. 1955. Ph.D. thesis. On file at the library, University of Washington.

The final, and most frequently overlooked, source of sediment is the seaward end of the estuary. Sediment brought to the mouth of the estuary by longshore movement and coastal currents can be carried into the area by tidal currents, wave-induced movement, the salt-replacing bottom current, and other elements of the general circulation. In some areas the mouth source of sediment is the dominant one and overshadows all others. In the major east coast estuaries it has been observed that the sand content increases from the head to the mouth, and this may be the result of introduction of sediment from a mouth source.

The importance of the mouth source is difficult to evaluate in any except special cases. Ippen [3] suggests that the salt-replacing bottom current may be of major importance in moving sands headward in some of the east coast major estuaries. In any area with abundant longshore and coastal current along the open coast outside the estuary, the importance of the mouth source must be considered.

The phenomena contributing to the transport of sediment that give some insight into the possible magnitudes of fluvial and marine processes are:

(1) River-associated flow;
(2) Tidal influence —
  (a) Vertical movement (tides), and
  (b) Horizontal movement (tidal currents);
(3) Wave influence —
  (a) Longshore and littoral transport or littoral transport,
  (b) Bottom movement under shoaling waves, and
  (c) Suspended movement under shoaling waves;
(4) General circulation —
  (a) Related to salt budget,
  (b) Related to tides,
  (c) Related to waves, and
  (d) Related to winds.

Seaward movement of river-introduced sediment by river current alone is of limited extent in any estuary. Without those aspects of turbulence generally associated with processes other than river flow, even the fine sediment that normally moves seaward in the surface layers would not be maintained in suspension long enough to bypass the estuary in any quantity. Bedload is characteristically deposited near the point of entry of the river into the estuary or near the point of maximum intrusion of saline bottom water.

The influence of tidal action is important both as a direct cause of sediment movement and in many indirect ways. The periodic diminishing and reversal of tidal currents allow much of the sediment in suspension to be deposited. If subsequent maximum velocities are too small to give impulse to these particles, their transport directly as a result of tidal currents is ended. In addition, the geometry of many of the nonmajor river estuaries is such that the tidal currents are generally oscillatory and result in little net movement. Those net movements that have been observed appear to be more closely related to the indirect effect of the tidal current on the general circulation (*1*).

Most other effects of tidal action are also indirect. The rise and fall of the water has no direct effect except that due to the periodic covering and uncovering of the nearshore bottom. The effect of the tidal action as a mechanism for mixing fresh and saline water exerts a control on the general circulation of the estuary.

One of the major marine influences that controls sedimentation in the estuary (and, specifically, in the nearshore areas) is the effect of wave action. The process of longshore transport is generally related either to tidal currents or to net movements under the influence of wave action. In contrast to the larger scale oscillatory movement associated with tidal currents, there will generally be, at any point that is under wave influence, a more pronounced unidirectional movement of the bottom sediment associated with wave action.

The direction of this transport associated with wave action is related to the direction of dominant wave propagation. Actual sediment movement may be generally with, or counter to, the direction of wave movement, depending upon factors associated both with wave and sediment particle dimensions and the depth of water.[4]

All the factors mentioned above are individually important, but they also contribute to the general circulation in the estuary. One element of the general circulation is the headward movement of bottom water resulting from continuity requirements imposed by the removal of salt (*7, 10*). The interaction of the tidal currents with the bottom results in a net mass transport as described by Abbott (*1*). Other mass transports that result from the net velocity profiles associated with shoaling waves (*8*) and from motion due to the effect of wind stress (*5*) contribute to the general circulation.

The general circulation, as defined here, is a low velocity net transport that should be of little direct importance in the transportation of

[3] IPPEN, A. T. SEDIMENTATION IN ESTUARIES. Mass. Inst. Technol. Class notes for course in estuarine environment. 1960. (Mimeo.)

[4] VINCENT, G. CONTRIBUTION TO THE STUDY OF SEDIMENT TRANSPORT ON A HORIZONTAL BED DUE TO WAVE ACTION. Sixth Conf. on Coastal Engin., Miami, Proc., pp. 326-355. 1958. Council on Wave Research, Univ. of Calif. (Berkeley).

sediment. However, combined with processes capable of giving impulse to, of resuspending or of maintaining sediment in suspension, the general circulation becomes a major factor in controlling the distribution and sorting of sediment within the estuary. Although an evaluation of the general circulation in any estuary is an almost impossible undertaking (*6*), the interaction of river flow, tidal currents, wave-related movement, and wind-induced current appears to be the basic control.

A brief discussion of some of the features of Port San Juan, British Columbia, may be used as an example of the importance of the mouth source and the marine processes. Although this is a small estuary of the Gordon and San Juan Rivers, fresh-water discharge appears to be adequate to allow development of a two-layered system. An evaluation of the possible sources of sediment has been conducted and indicated that the shoreline, bottom, and surface sources may be considered negligible.[5] The introduction of a characteristically micaceous sediment by the rivers is indicated by a very localized occurrence associated with the extreme head of the estuary. On the other hand, the mouth source is considered of primary importance, since the net longshore movement of sediment along the coast outside Port San Juan provided material of the same general characteristics as that collected from the bottom over most of the area. This sediment is mineralogically distinct from the river-related micaceous sediment mentioned above.

In Port San Juan, it is impossible to separate the processes of initial transportation and deposition from processes of reworking. The unimodal, well-sorted sediment and the lack of indications of permanent deposition imply that a reworking after initial deposition is an important and continuing process.

The major contribution of the river to the transport of sediment appears to be in relation to the role of fresh-saline water mixing in influencing the general circulation. This general circulation appears inadequate to give impulse to any sediment particles on the bottom, but should act as an initial trap for any sediment introduced from the river source.

The effect of longshore movement and sediment sorting under wave action appears to be of primary importance in Port San Juan. Measurements conducted by the National Research Council of Canada indicate that a swell having an 8- to 10-second period enters the mouth of the estuary throughout most of the year.[6] Waves of this magnitude are capable of giving impulse to sedimentary particles over the whole of the estuary bottom because of its relatively shallow depth (18 meters and less).

The estuary acts as an effective trap for much of the river-introduced sediment, but the absence of the finer sizes raises the possibility of bypassing of the fine sediment as the result of interaction between the wave net-velocity profile and the sediment-suspension gradients of the finer particles.

With the addition of sediment from both the mouth and the river, there remains the problem of satisfying the continuity requirements. Because the waves are capable of giving impulse to bottomed particles, the mode of movement induced by wave action is a function of the size of the wave and the normal daily and seasonal variations in wave dimensions will impose varying effects. The overall bed movement will always be in the direction of the dominant wave propagation, headward in this estuary. The suspended load will be affected differently as the wave dimension modifies with passage over a shoaling bottom.

Under maximum wave conditions there may even be a net removal of sediment from the area. It appears more probable, however, that the amount of bedload entering from the mouth is compensated by a continuing removal of the finer sediment of stream origin and a periodic removal of some of the sand sizes from the bottom. If it can be assumed that the amount of river-introduced sediment is a function of the river discharge, the rate of addition of sediment by streams is greater for the winter months during the very time the larger waves are generally steeper but less persistent. During the summer, the amount of stream-introduced sediment is sharply decreased at the same time that the waves entering the area are most persistent. The winter waves appear to be capable of removing greater peak loads and coarser sediment during the times when they are effective in the area; the summer waves, because of their greater persistence, probably remove a greater overall volume of sediment, but without the coarse extremes.

Additional field testing of the general model of sedimentation in some nonmajor river estuaries has verified the importance of the nonriver sources of sediment. Further, there is good evidence that most sediment introduced into estuaries, from whatever source, is subject to an initial deposition within the limits of the area. An understanding of the processes capable of providing impulse to these bottomed particles is

[5] BURNS, R. E. A MODEL OF SEDIMENTATION IN SMALL, SILL-LESS, EMBAYED ESTUARIES OF THE PACIFIC NORTHWEST. 117 pp. 1962. Ph.D. thesis. On file at the library, Univ. of Wash.

[6] NATIONAL RESEARCH COUNCIL OF CANADA. PRELIMINARY (WAVE) STUDIES. 4 pp. and 4 figs. 1961. Division of Mechanical Engineering. Unpublished manuscript on Port San Juan.

necessary in order to evaluate the eventual distribution of sediment. In the nonmajor river estuaries these processes are rarely related directly to river flow, but appear to be predominantly of marine origin.

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HOPPER DREDGE DISPOSAL TECHNIQUES AND RELATED DEVELOPMENTS IN DESIGN AND OPERATION

[Paper No. 65]

By LOUIS J. MAURIELLO, *chief, Marine Division, U.S. Army Engineer Division, North Atlantic, New York, N.Y.*, and LEWIS CACCESE, *chief, Operations Division, U.S. Army Engineer District, Philadelphia, Pa.*

Synopsis

A considerable amount of dredging by the Corps of Engineers to improve and maintain navigation channels in various regions of the United States is accomplished by seagoing hopper dredges. Conventional methods of operation are described. Various disposal techniques employed, particularly recent developments, and related modifications in design and operation are discussed.

Introduction

A significant amount of dredging performed by or under the jurisdiction of the Corps of Engineers, U.S. Army, requires the use of seagoing hopper dredges. The Corps presently owns and operates in the various coastal and Great Lakes regions of the United States 15 dredges of this type with hopper capacities ranging from 500 cubic yards to over 8,000 cubic yards. Physical and other descriptive data of this hopper dredge fleet is shown in table 1. The annual total cost of operating the dredges amounts to nearly $19,000,000, with costs of individual dredges ranging from approximately $700,000 to $2,400,000 a year. In recent years, the total volume of material removed annually by these dredges is approximately 65 million cubic yards, as measured in place within project limits.

Basically, the seagoing hopper dredge is a self-propelled vessel resembling the modern ocean tanker in appearance except for a larger amount of deck equipment (figs. 1 and 2). In lieu of the cargo tanks of the ocean tanker, the hopper dredge is provided with hoppers or bins used to load and carry material dredged hydraulically from the bottom. It is a completely self-contained dredging plant equipped with all necessary dredging equipment (i.e., centrifugal pumps, dragarms or trailing suction assemblages, etc.). The significant characteristic of the hopper dredge is that it operates while underway and requires no anchors or other mooring devices. Modern hopper dredges reflect about a century of development. They usually work in channels or harbors in which wave action or heavy traffic makes a stationary dredging plant undesirable or intolerable; also, in many other waterways the location of the shoal and the disposal area are so far removed from each other that the mobility of the hopper

TABLE 1.—*Specifications of Corps of Engineers hopper dredges*

Name	Year built	Length, beam, and depth	Maximum hopper capacity	Maximum draft loaded	Propulsion, All twin screw		Dredge pumps			Remarks
					Horsepower	Type [1]	Number	Size	Horsepower	
		Feet	*Cu. yd.*	*Ft. In.*				*Inches*		
Essayons	1950	525x72x40	8,270	31–0	8,000	TE	2	32	1,850	
Goethals	1938	476x68x36	6,422	29–0	4,500	TE	2	30	1,300	Equipped for sump rehandling.
Biddle	1947	352x60x30	3,060	24–3¾	6,000	TE	2	28	1,150	
Comber	1947	352x60x30	3,422	24–3¾	6,000	TE	2	28	1,150	Equipped for direct pump-out.
Gerig	1947	352x60x30	3,060	24–3¾	6,000	TE	2	28	1,150	
Langfitt	1947	352x60x30	3,060	24–3¾	6,000	TE	2	28	1,150	
Harding	1939	308x56x30	2,682	20–3	4,240	D	2	20	650	
Markham	1960	339x62x28	2,681	20–0	5,300	DE	2	23	[2] 650	Equipped for direct pump-out.
Mackenzie	1924	268x46x22	1,656	21–0	2,400	DE	1	26	900	
Hains	1942	216x40x15	885	13–0	1,400	DE	1	20	410	Equipped for direct pump-out and side-casting.
Hoffman	1942	216x40x15	920	13–0	1,400	DE	1	20	410	
Hyde	1945	216x40x15	720	13–0	1,400	DE	1	20	410	
Lyman	1945	216x40x15	920	13–0	1,400	DE	1	20	410	
Davison	1945	216x40x15	720	13–0	1,400	DE	1	20	410	
Pacific	1937	180x38x14	500	11–3	1,200	D	1	18	340	

[1] TE = Turbo-electric; DE = Diesel-electric; D = Diesel direct drive.
[2] Also equipped for operation at 1,000 hp.

FIGURE 1.—Hopper dredge *Essayons*.

dredge makes it by far the most efficient dredging plant.

As in any dredging operation, the location of the dredging site, the character of the bottom material, the regimen of the waterway, and the availability of suitable disposal areas are important factors that affect the economics of hopper dredging and dictate the methods and techniques employed. The principal intent of this paper is to discuss the influence of such factors on hopper dredge disposal techniques and design.

FIGURE 2. — Hopper dredge *Markham*.

How Hopper Dredges Usually Work

Although this paper will deal particularly with the disposal phase of hopper dredge operations, a brief discussion of how hopper dredges normally work and certain elements involved in their operation is appropriate.

Hopper dredges are designed primarily to dredge materials hydraulically, to load or retain the solids dredged in their hoppers, and then to haul the spoil to a disposal area or dump, where the material is disposed of by dumping through doors in the bottom of the hoppers (fig. 3). Loading is accomplished by sucking the bottom material through the drags into the hoppers while making one or more cuts (or passes) through the dredging area. The quantity or volume pumped during the loading operation depends primarily upon the character of the material and the amount of pumping time involved as well as the hopper capacity and the pumping and propulsive capability of the dredge being used. The length of haul also affects the amount of time that may be spent pumping. During the dredging (or pumping) process the bottom material is pumped in a diluted state and the hoppers are equipped with overflows to allow excess water to be discharged overboard. In the case of such materials as very heavy coarse sand and gravel that readily settle in the hoppers, most, if not all, of the solids are retained in the hoppers and only minor quantities of solids are lost in the overflow. Conversely, in certain materials such as very light and fine silt, most of the solids pumped after the hoppers are filled with the dredged mixture may be overflowed if pumping is continued. Generally, the preponderance of materials encountered in dredging problems fall in between these extreme categories. For illustrative purposes, figure 4 shows theoretical hopper dredge loading curves for materials of various degrees of retainability between 0 and 100 percent, if a uniform solids pumping rate is assumed. The retainability factor obviously dictates to a large measure the time involved in loading and the hopper load obtained. In addition the length of haul and the time spent in making the round trip to and from the disposal area and dumping the load are important in determining when to terminate loading so as to attain the so-called "economic load" and "economic pumping time." The economic load and economic pumping time are determined by a series of economic load tests conducted periodically during normal dredging and hauling operations. During these test loads, measurements of the hopper load are made when it reaches overflow level and thereafter at least at every turn; the economic load is dredged when the retained cubic yards per minute of total cycle time (i.e. pumping, turning, running to and from dump, and dumping) equals the incremental retained

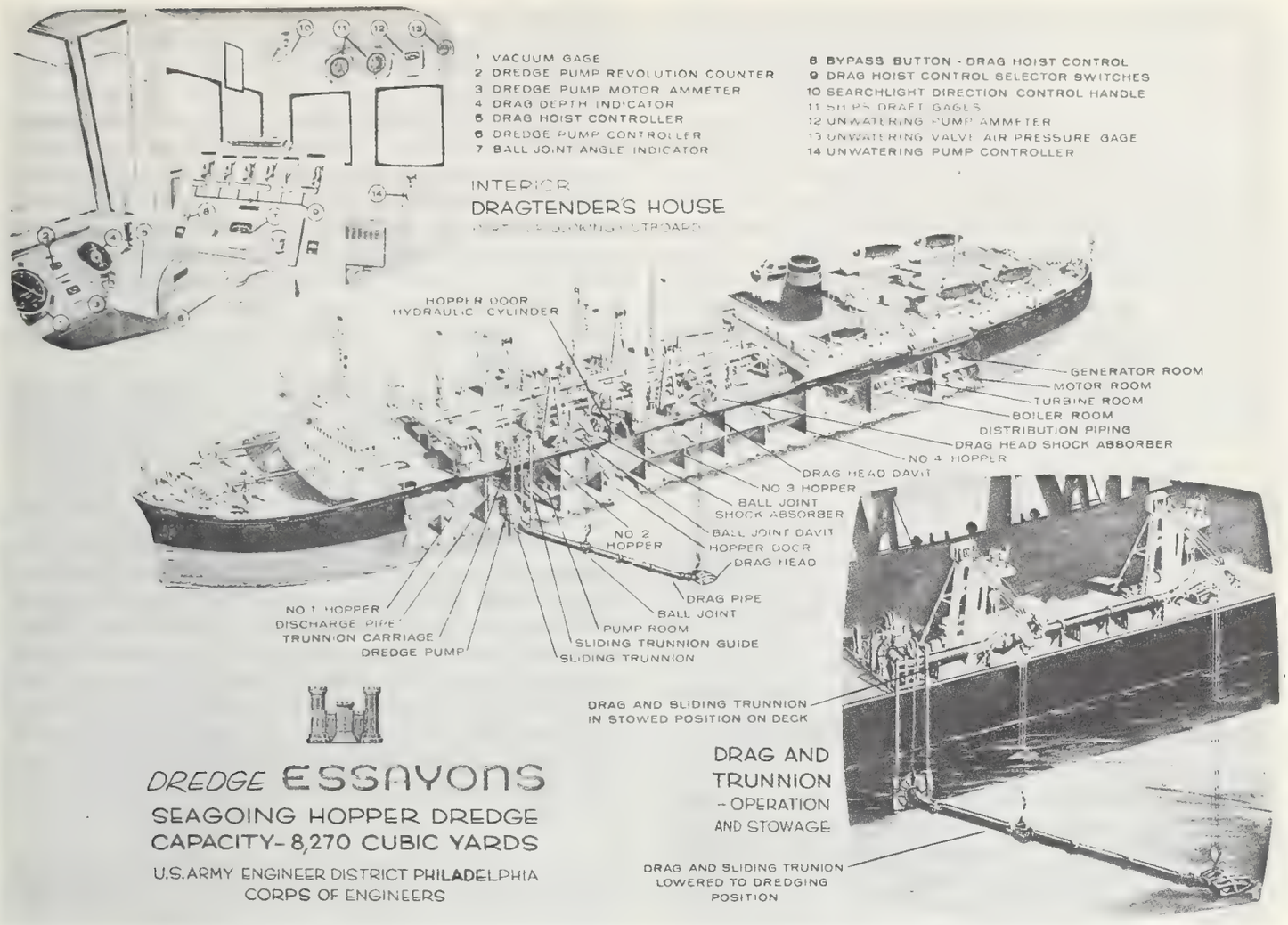


FIGURE 3. — Cutaway view of hopper dredge *Essayons*.

cubic yards per pumping minute or expressed in mathematical form when:

$$\frac{\Delta Y}{\Delta t} = \frac{Y}{T+t} \qquad \text{where}$$

Y = Hopper load measured in cubic yards of material comparable in density to that in situ;

ΔY = Increase in hopper load between measurements, in cubic yards;

t = Pumping time, in minutes;

Δt = Pumping time to obtain increase in hopper load (ΔY) between measurements, in minutes;

T = Nonpumping time (i.e., turning, running to and from dump, and dumping), in minutes.

During initial stages of loading prior to reaching an economic load, $\frac{\Delta Y}{\Delta t}$ invariably exceeds $\frac{Y}{T+t}$ and this is indicative of underpumping. As loading progresses $\frac{Y}{T+t}$ normally increases and $\frac{\Delta Y}{\Delta t}$ decreases and approach each other in value. If pumping continues after an economic load is obtained, then $\frac{\Delta Y}{\Delta t}$ becomes less than $\frac{Y}{T+t}$, which indicates overpumping.

In practice the point of economic loading generally is not exactly defined for each and every load dredged, since it may well occur at a point other than exactly at the end of a dredging cut. Therefore, overpumping to a slight degree usually results. This is generally true except in cases where loading by pumping only to overflow is found to be economical or otherwise desirable. In such cases, since essentially nonretainable materials are involved, the hoppers are normally dewatered before commencement of loading.

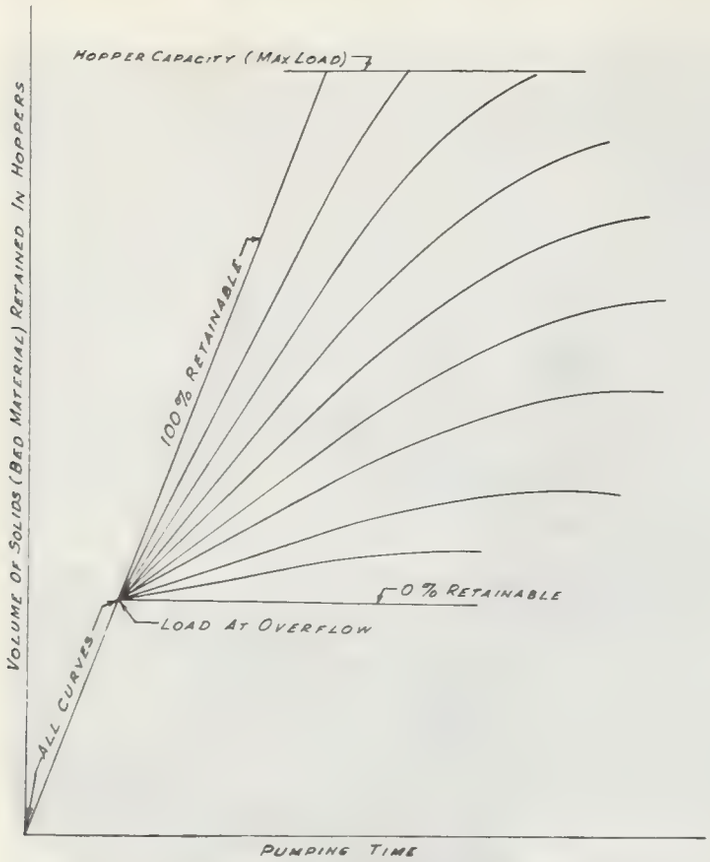


FIGURE 4. — Retention in hoppers of varying grain-sized materials.

Upon completion of loading, the dredge proceeds at full speed to a designated disposal area to dump the load. After disposal, the dredge returns to the dredging area to repeat the loading.

Disposal by Conventional Dumping

As indicated above, hopper dredge spoil is normally disposed of by dumping through doors in the bottom of the hoppers. Emptying the hoppers by this operation takes about 3 to 15 minutes, depending principally on the volume and nature of the material in the hoppers. Generally, relatively fluid materials are dumped readily, whereas sticky clays and certain granular materials are more difficult to dump and washout and hopper jetting may be required. In this regard, it has been found that the dumping of heavy medium sand of essentially uniform grain size existing in various coastal regions may induce severe hull vibrations, unless the dumping sequence of individual hoppers is carefully controlled and large amounts of flushing and jetting water are used.

In most cases, hopper dredge spoil is presently being disposed of by dumping in designated areas in deep water at sea or outside navigation channel limits in open bays or rivers. Dump areas usually are selected with a view toward effecting permanent (within practical limits) disposal of the dredged material while maintaining the length of haul to a minimum. Application of these criteria results in compromises based on practical considerations of the particular job conditions and the economics involved. In general, the hauling distance for Corps of Engineers hopper dredging averages from 3 to 4 miles. The degree of harbor waterfront industrialization in certain waterways makes it necessary, however, to haul spoil considerably greater distances. For example, all spoil dredged by hopper dredge from principal channels in New York Harbor is dumped in a designated area of deep water in the Atlantic Ocean, involving a haul of more than 30 miles from certain locations.

Practically all developments leading to the use of hopper dredge disposal techniques differing from conventional dumping in deep water, have stemmed primarily from efforts to reduce the length of haul required for disposal.

Dumping in Rehandling Basins

In operating hopper dredges in the Delaware River from Philadelphia to the sea, a haul of 50 miles or more would be required to effect disposal in deep water at sea. The necessity for precluding such long hauls was realized as early as 1880, when provisions were made for hopper dredges to dump in prepared basins from which a pipeline dredge redredged the material and discharged it ashore behind retaining dikes. The basin was in effect a previously dug hole in the riverbed into which materials were dumped by the hopper dredge. The rehandling dredge then pumped this material from the hole to the shore disposal area. This scheme, with improvements from time to time to reduce losses of spoil dumped by the hopper dredge before it could be picked up by the rehandling dredge, was used extensively in the Delaware River for over a half century. The rehandling basins were usually located in pairs so that one could be used by the hopper dredge to dump into while the other was being emptied by the rehandling pipeline dredge. A typical arrangement is illustrated in figure 5. Maneuvering the hopper dredge in extremely restricted basins requires considerable care and greater skill by navigating personnel than in conventional dumping in open deep water. Therefore, the time required for dumping in such operations usually was greater than in the latter case.

The bottom material comprising recurring shoals in the lower part of the Delaware River is a light silt, about 1,300 grams per liter in density, with approximately 90 percent by weight of the solid grains finer than 0.06 mm. in diameter. The retention of such material in rehandling basins presents a problem. In the Delaware River it was found that at least half the volume of material dumped by the hopper

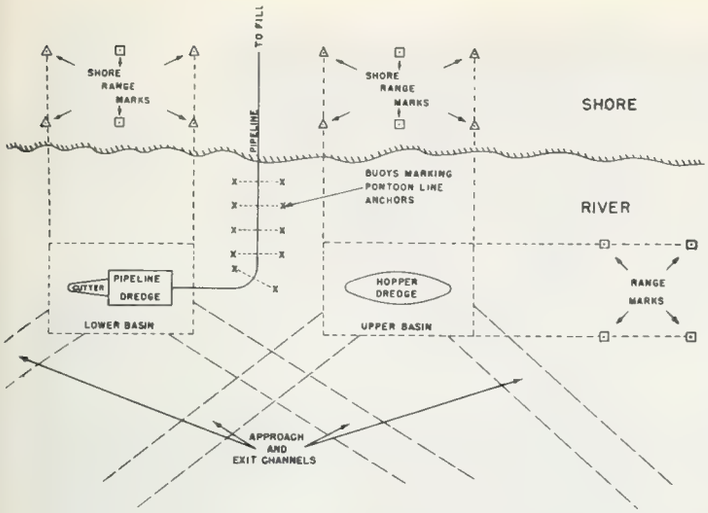


FIGURE 5. — Typical rehandling scheme.

dredge was carried away by tidal currents before it could be picked up by the pipeline dredge. As a result this scheme was discarded in 1954 by adoption of the sump rehandling technique, as described hereafter in this paper. It is considered, however, that the method of rehandling spoil by dumping into open water basins has merit when the material is heavy and consists of relatively coarse grains that readily settle after dumping and are not significantly affected by prevailing currents.

Agitation Dredging

During the loading of hopper dredges, as described above, the overflow contains varying amounts of solids that are returned to the waterway. A significant part of this overflowed material, particularly when fine-grained and slow-settling, may be carried and distributed by currents over wide areas in the vicinity of the dredging site. In a strict sense, this constitutes "agitation dredging." Some degree of agitation exists in most types of dredging and it is fundamentally inherent but not intentional in most hopper dredge operations. However, by definition, agitation dredging by hopper dredge is a process that intentionally discharges overboard large quantities of dredged material with the objective that a major part will be transported and deposited outside channel limits by tidal, river, or littoral currents. This method of dredging with direct disposal or discharge in the immediate vicinity of the area being dredged may be advantageous when most of the material handled does not resettle in the channel and is not deposited where it can become a potential source of detrimental shoaling that may require future excavation. A fairly thorough knowledge of prevailing currents in the waterway and the composition and characteristics of the sediment to be dredged is essential to determine the feasibility of employing agitation dredging of any kind. Conditions may be favorable for agitation dredging when the shoal material consists of fine grains in the silt and clay categories with extremely slow settling rates, and dispersal of the dredging material over a large area is affected by prevailing currents. A chart developed and used by the Corps of Engineers to determine the largest grain size that may be expected to reach a deposit area of given depth at a given distance from the dredge is shown in figure 6. In any event, in specific cases, although favorable field conditions for this method of dredging may be indicated, the effectiveness of the operation also must be evaluated not only by the immediate and lasting improvements in the particular channel dredged but also with the full consideration of possible adverse effects in adjacent areas as well.

In agitation dredging, the hopper dredge normally continues to make repetitive cuts through the dredging area while pumping well beyond economic loading limits. The hauling and dumping phases of operation are ordinarily eliminated except for occasional trips to the dump to avoid consolidation of material in the hopper that may eventually result in undue difficulty in dumping. In some cases, to avoid consolidation, the dump doors may be kept cracked open and lower (emergency) overflow gates may be used for discharging the material overboard. Also, at certain locations where agitation dredging is not feasible during parts of the tidal current cycle (i.e. slack water, floodtide), dredging and hauling on an economic basis are employed intermittently.

Corps of Engineers' hopper dredges have been used with mixed success for agitation dredging in practically all regions of the United States since the early 1900's. In some cases, it has been found to be an expedient and economical means of keeping navigation channels open. There has been a decided decline in the use of this dredging technique by hopper dredges since 1955. The big advantage in agitation dredging is the ability to move large masses of material from shoal areas quickly and cheaply. It therefore cannot be overlooked as a dredging technique where it can be successfully employed. In some cases, particularly where frequent recurring dredging requirements are present regardless of the dredging methods and type of plant used, any disadvantages of this procedure can be tolerated with good judgment for the economic considerations involved. At present, agitation dredging is occasionally used in certain entrance channels along the Gulf coast, such as the Southwest Pass of the Mississippi River in Louisiana, and the Mississippi Sound channels leading into Gulfport and Pascagoula

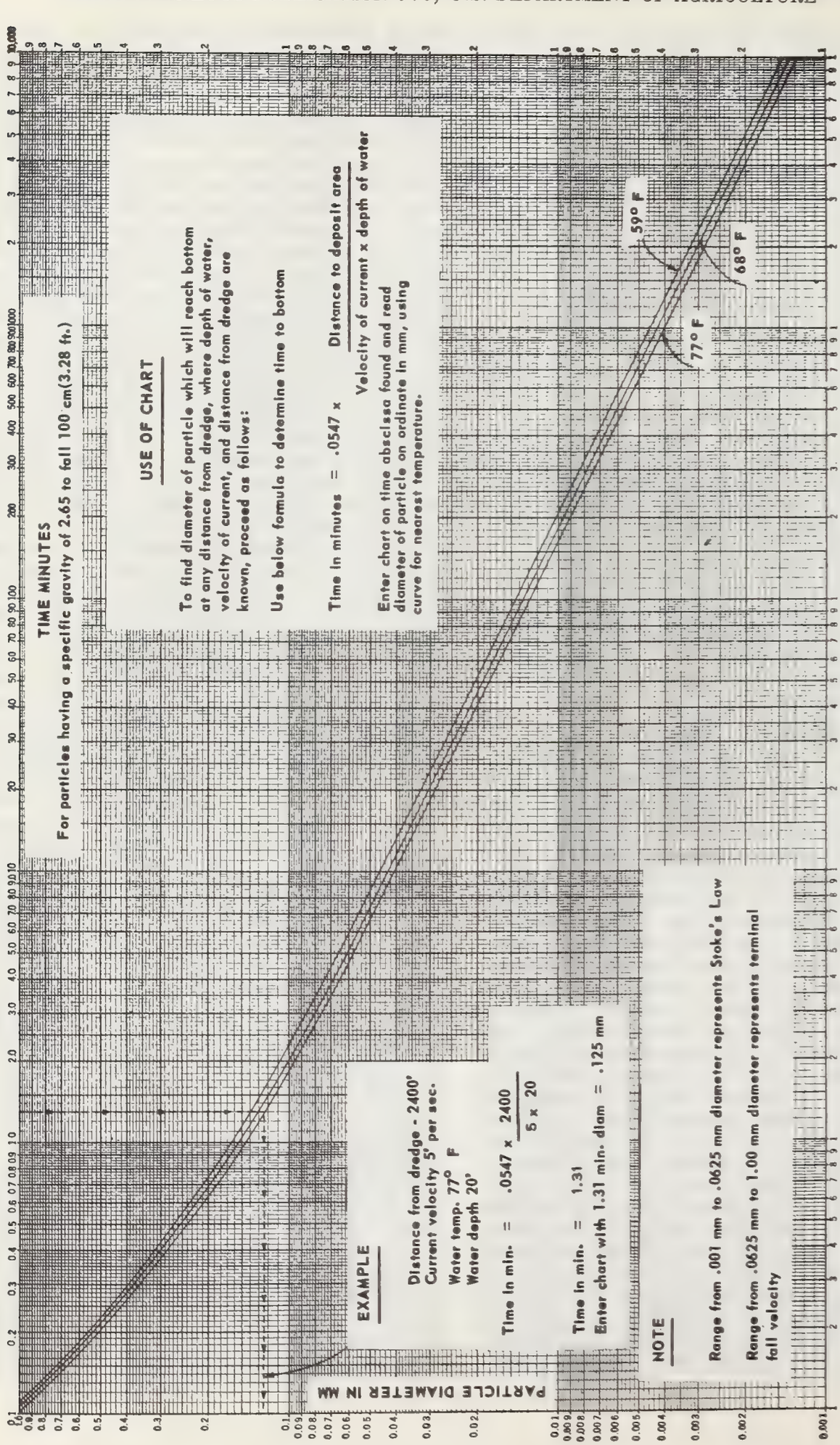

TIME MINUTES
For particles having a specific gravity of 2.65 to fall 100 cm(3.28 ft.)
USE OF CHART
To find diameter of particle which will reach bottom at any distance from dredge, where depth of water, velocity of current, and distance from dredge are known, proceed as follows:
Use below formula to determine time to bottom
Time in minutes = .0547 x Distance to deposit area / Velocity of current x depth of water
Enter chart on time abscissa found and read diameter of particle on ordinate in mm, using curve for nearest temperature.
EXAMPLE
Distance from dredge - 2400'
Current velocity 5' per sec.
Water temp. 77° F
Water depth 20'
Time in min. = .0547 x 2400 / 5 x 20
Time in min. = 1.31
Enter chart with 1.31 min. diam = .125 mm
NOTE
Range from .001 mm to .0625 mm diameter represents Stoke's Law
Range from .0625 mm to 1.00 mm diameter represents terminal fall velocity
59° F
77° F
68° F
PARTICLE DIAMETER IN MM

FIGURE 6. — Chart for determining largest grain size to reach deposit area.

Harbors in Mississippi. In these projects, the dredge *Langfitt* has agitated and removed a million or more cubic yards of sediment from within channel limits within a 1-week dredging assignment. The annual total volumes agitated by hopper dredges in the past 10 years has ranged from 1.3 to 11.3 million cubic yards.

Sump Rehandling Operations

A review of the amount of dredging effort required to maintain navigation projects dredged by hopper dredges was made in 1950-52. This review revealed the need for developing new methods for the disposal of hopper dredge spoil. The Delaware River, below Philadelphia, was an example of a location where costly maintenance was resulting from material that was being lost in the dredging process. Engineering studies indicated that approximately 20 to 25 million cubic yards of material were being dredged each year in the maintenance of this channel, whereas only approximately one-third of this amount was entering the estuary as new material. After studying the problem, it was concluded that the difference between the total volume of material being dredged each year and the estimated sedimentation or natural shoaling represented the quantity of material that was escaping from open water disposal areas after the dredged materials had been dumped from hopper dredges.

Based on this conclusion, it was decided that a method of providing for the positive capture, retention and permanent disposal of dredged material was required to effect a significant reduction in the annual dredging workload. Experience had demostrated that agitation dredging, bottom dumping in open waters, or bottom dumping in rehandling basins for removal by pipeline dredges did not offer any degree of reasonable success in reducing the annual volume of maintenance dredging by hopper dredges. Various possible solutions were studied, and from this study there evolved the idea of unloading a hopper dredge through a self-contained and elevated discharge line into a rehandling barge or vessel, located at selected suitable mooring stations, from which the material could then be pumped ashore. Since the material to be dredged in the Delaware River was predominantly light silt susceptible to being transported extended distances by the river currents to possibly form new shoals, it was also decided as a general rule, that the hopper dredges would load by pumping only to the overflow level and then haul and dispose of the material into the rehandling vessel.

Conversion of a hopper dredge to unload into a rehandler included the installation of a collecting system in the hopper area to transport the dredged materials in the hoppers for delivery into the rehandler through an elevated pipe (referred to as a "snorkel") that is extended over the dredge side (fig. 7). This system, including the necessary valves, permits the discharge of material into the rehandler by a hopper dredge, using the same pumps utilized for dredging. For example, on the dredge *Goethals*, two 32-inch collecting mains with suction inlets into the hoppers extend longitudinally from the rear to the forward end of the hoppers. These lines are connected to the suction side of the two 32-inch dredge pumps. The material is then pumped into a single 42-inch pipeline at the main deck level and through the discharge snorkel into the rehandler. A sump rehandling vessel was provided by converting an obsolescent hopper dredge (the *New Orleans* built in 1912) and using its hopper area as the storage compartment. The propulsion and original dredging machinery was removed and a 6,000-hp. pump installed for discharging the material from its hoppers to shore disposal areas in the same manner as a pipeline dredge (figs. 8 and 9).

Since December 1954 the sump rehandling dredging system has been used with considerable success to maintain a major part of the 96-mile channel of the Delaware River between Philadelphia and the sea. It has also been used to improve and maintain the Norfolk Harbor channel. From December 1954 through June 1962 the following quantities have been handled utilizing this technique.

	Cubic yards
Delaware River	77,390,700
Norfolk Harbor	10,908,200
Other areas	2,782,000

It has been found that positive disposal and retention of material which has formed shoals can be obtained at reasonable costs. The material when placed behind shore disposal area dikes is not only removed as a potential source of future shoaling but also frequently enhances land values.

The measure of success of the sump rehandling system with its criteria for positive disposal of all dredged material is manifested by the Delaware River maintenance history. Prior to sump rehandling, as much as 25 million cubic yards of material were being handled annually by hopper dredges to maintain the Delaware River channel. This workload required as many as 22 hopper dredge months of operation per year, together with the full-time use of several pipeline dredges to rehandle material ashore. Presently, the lower Delaware River channel is being maintained by removing approximately 7½ million cubic yards annually, involving the use of about 10 hopper dredge months. Despite this reduced effort, a better channel exists. In brief, the development of the sump rehandling

FIGURE 7. — Hopper dredge *Goethals*, showing 42-inch discharge pipe for sump rehandling.

system and the positive disposal of dredged material has reduced the required dredging effort to approximately one-half of its former level.

It was found that the sump rehandler was the answer where:

(1) The particle sizes of the sediment are predominantly in the silt category or finer. (In the Delaware River it is most common that less than 10 percent of the shoal material is in the sand category.)

(2) Enclosed disposal areas can be located at a distance of within 3 miles of the sump rehandler.

(3) A mooring for sump rehandler can be provided in 30 feet of water to permit the hopper dredge to maneuver and moor for unloading.

(4) It is determined that modern pipeline dredges cannot be used effectively because of any of the following conditions:

(a) Distance between the shoal and disposal area exceeds a reasonable discharge pipeline length;

(b) Water and weather conditions are too extreme or severe; or

(c) Floating pipeline is too obstructive to channel navigation.

The above criteria all existed in the lower Delaware River and Philadelphia Harbor and as a result the sump rehandler was successfully operated.

A significant factor in employment of the sump rehandler is that by the nature of the operation the maximum economy results when both converted hopper dredges — that is, the *Comber* and the *Goethals* — are used in conjunction with the sump rehandler *New Orleans*. This results in large production where as much

FIGURE 8. — Sump rehandler *New Orleans.*

as 2 million cubic yards of shoal material are placed ashore per month and positively retained behind banks.

The sump rehandling team of two hopper dredges and a sump rehandler dredges mass quantities of material efficiently and effectively; however, three major pieces of dredging plant are required. Not many projects either have the requirement for so much capacity or are able to justify the mobilization and operating cost of three major pieces of plant. This has been the major limitation that prevents the widespread application of this technique beyond the principal waterways of Philadelphia and Norfolk.

From the sump rehandling experience it was apparent that positive retention of dredged material is the best solution to many dredging requirements. However, the fact that the relatively high operating cost of the sump rehandler system precluded its use on projects with limited dredging requirements or workload necessitated further studies to provide additional improvements in disposing of hopper dredge spoil. As a result the design of the direct pumpout system for hopper dredges was developed to overcome this limitation. This direct pumpout system was devised to accomplish the same positive retention of dredged material as the sump rehandling system but with only one major item of plant. It is anticipated that direct pumpout capability will bring the technique of positive disposal of hopper dredge spoil within the financial capacity of many more projects.

Direct Pump Ashore Operations

The sump rehandling system experience established that the positive disposal of all dredged spoil is an excellent solution in projects with dredging requirements such as the Delaware River. It was also evident that this dredging system was compatible with all campaigns for cleaner waters and most consistent with the Corps of Engineers responsibilities for assuring that solids are not indiscriminately introduced into the navigable waters of the United States. These observations, together with the concern for reducing the high operating cost of the sump rehandling operation, led to consideration of equipping the hopper dredge to con-

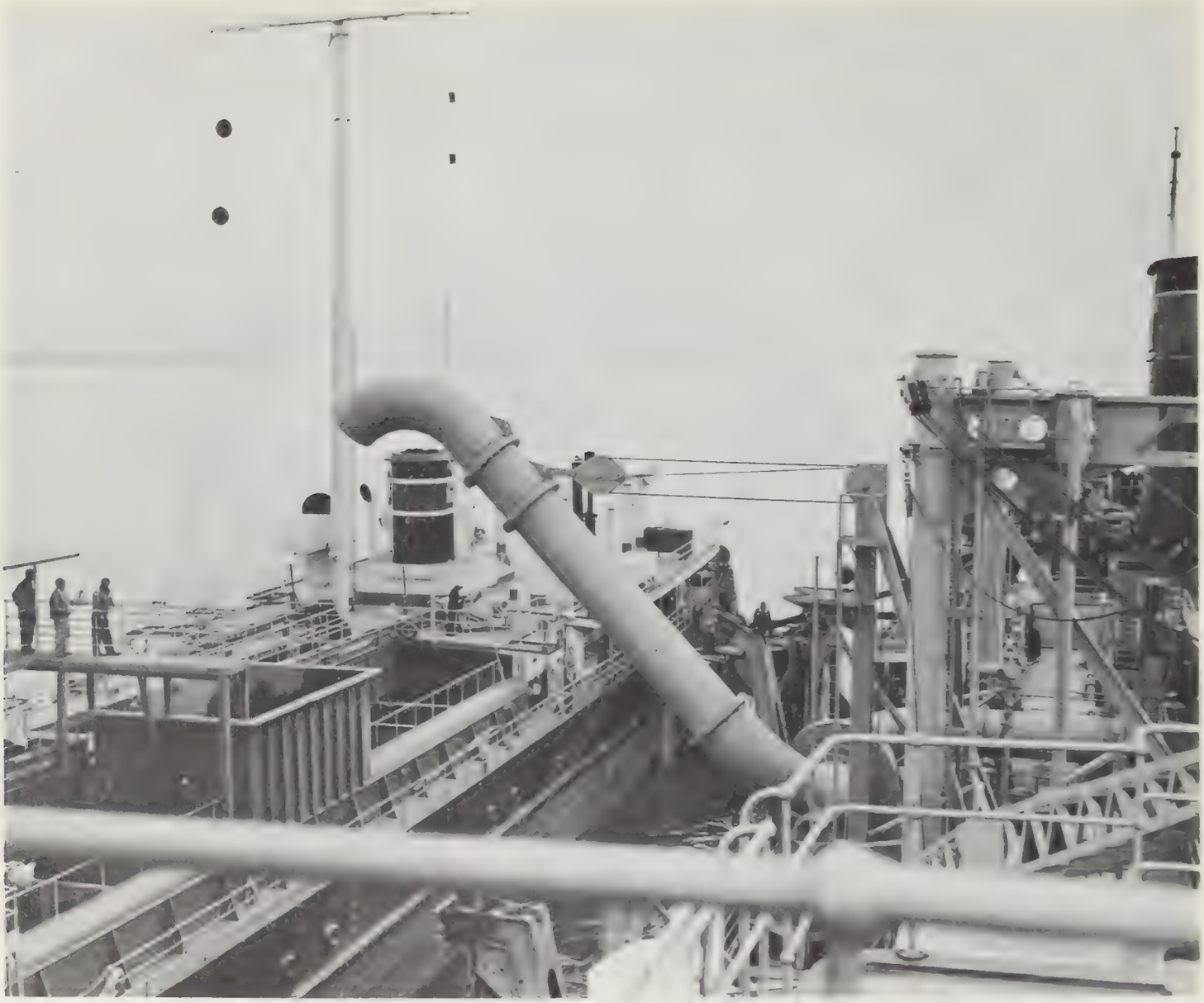


FIGURE 9. — Hopper dredge discharge being set into receiving hopper of sump rehandler *New Orleans*.

nect directly to a pipeline and pump itself out. Obviously this procedure would slow down the disposal operation of the hopper dredge, but a compensating feature would be the elimination of a high-cost rehandling plant required in the sump rehandling process.

Direct pumpout of Government hopper dredges had not been previously used to any appreciable extent. The hopper dredge *New Orleans* was equipped with a direct pumpout system when originally constructed, but no record of the use or effectiveness of this system is available. The hopper dredge *San Pablo* did do some direct pumpout dredging in 1923 and 1924 at Mare Island Strait with an improvised system, but again the records do not indicate any evidence of the degree of its success.

The first modern effort to pump out directly a Government hopper dredge was with the *Hains* in 1959. The direct pumpout capability on the *Hains* was obtained by improvised modifications to the piping system that is normally used to de-water the hoppers. This system normally removes all water that remains in the hoppers after dumping, so that the dredging may commence with completely empty bins. The dewatering system was modified so that the dredged material could be sucked from the hopper and then pumped through a pipe connected to shore. This first effort was on the Calumet River, where it was formerly necessary to haul dredged material 13 miles into Lake Michigan. More recently, direct pump ashore with the *Hains* also has been utilized in dredging the Rouge River. An area of 75 acres at Grassy Island in the Detroit River has been diked and

riprapped and mooring facilities have been provided for direct pumpout operation. During the first two seasons of operation, over 300,000 cubic yards of material have been directly pumped ashore by the *Hains*. The average haul to the diked area is 6 miles, whereas the nearest alternate location for open water disposal in Lake Erie would have involved a haul of 23 miles. This success has prompted the Corps to make plans to procure and dike an area at Holland Harbor, Mich., to replace the long haul now necessary from the inner end of the project.

The trend toward more extensive use of this method of positive disposal of dredged material is next evident in the *Markham*. The original design and construction of this 2,600-cubic-yard hopper dredge, which was completed in 1960, included a direct pumpout capability. The dredge can place 2,000 hp. of its total available 5,300 hp. for use in the pumpout operation. This is in contrast to the dredge *Hains*, which is limited to only the 410-hp. pumping power available for normal dredging operations.

One pump ashore project was done by *Markham* in 1961 at Toledo, Ohio, where approximately 416,000 cubic yards of maintenance dredging material was pumped about 1,400 feet into a diked area provided by the city of Toledo on park property. A second shore disposal area is being developed at Toledo known as Buckeye basin, an inland area about 3,500 feet distant. The discharge line will be laid in an 8-foot storm sewer, which discharges into the Maumee River close to present moorings. Use of storm sewer is necessary because the area to be traversed is in the metropolitan part of the city. The Buckeye basin will hold about 5,000,000 cubic yards and it is estimated it will be adequate for about 20 years of maintenance dredging in the river section. A third shore disposal area in Toledo is being created just outside the river mouth by a new work contract requiring diking of a disposal area in Maumee Bay for pipeline dredging. This will be available in future years for hopper dredge disposal from the inner bay channel and lower river channel.

In a short time an area in Lake St. Clair known as Gull Island, a former open water pipeline dump when channels in the vicinity were dug, will have to be diked for confined disposal of maintenance dredging from channels in Lake St. Clair. The island is about 30 acres of firm ground with much additional bottom just awash around it.

From the knowledge that direct pump ashore operations have been performed successfully, coupled with an indicated need for replacing the existing sump rehandler *New Orleans* in the near future, it has been determined that a more efficient program would result from adoption of the direct pumpout technique on hopper dredges utilized in the Delaware River and Norfolk Harbor. A hopper dredge that can also rehandle material directly ashore has the obvious advantage of more mobility and less fixed operating cost than the system that requires a hopper dredge and a sump rehandler. As stated earlier, these latter features were the ones that were proving a limitation on a greater use of sump rehandling. In view of this, it was concluded that the method of direct pumpout was generally preferable and the decision was made to convert the *Comber* and *Goethals* for self-unloading directly into shore disposal areas and to eliminate the sump rehandler.

In summary, the decision for direct pumpout in lieu of sump rehandling was arrived at because:

(1) Direct pumpout plant (by conversion of existing hopper dredges) can be provided at less initial cost than sump rehandling plant.

(2) The volumes of material that can be placed through a sump rehandling facility indicates it may become limited to an inefficient part-time use.

(3) The lesser operating cost of a direct pumpout dredge than a hopper dredge plus a sump rehandler will place the means of taking advantage of a positive retention dredging system more within the budget limitation of smaller projects, which have not been able to afford the sump rehandler.

Sump rehandling ceased in February 1963 when the *New Orleans* was deactivated for eventual disposal. The hopper dredge *Comber* was converted for direct pumpout. This conversion alters the dredge so that her full propulsion power of 6,000 hp. is utilized for direct pumpout. The *Comber* pumpout power for sump rehandling was limited to the dredge pump power of 1,150 hp. per pump or 2,300 hp. total. This alteration provides the *Comber* with the ability to pump through 20,000 feet of pipeline. As a result, direct pumpout operations in the Delaware River were initiated in May 1963 and hereafter it is anticipated that the 9,000,000 cubic yards of material previously dredged annually with sump rehandling will be dredged, utilizing the direct pumpout technique instead. In order to provide the productive capability required to handle this volume of material, the dredge *Goethals* will also be similarly converted to include pump ashore facilities in the spring of 1964.

There is presently under design a new hopper dredge to replace the *Mackenzie* in the Southwestern Division. This proposed 3,100-cubic-yard dredge, to be named *McFarland*, will be the most modern that our present technology can produce. A feature of this dredge will be its

ability to utilize as much as 5,600 hp. for direct pumpout operations. This will enable the dredge to pumpout through as much as 20,000 feet of pipeline. This capability for direct pumpout ashore through a pipeline is expected to be of considerable value in certain areas of the Sabine-Neches Waterway and the Houston Ship Channel, particularly where a shortage of disposal areas sufficiently accessible for conventional pipeline dredges is developing as a result of waterfront industrialization.

Side-Casting (or Boom-Discharging) Operations

As indicated in the foregoing discussion on agitation dredging, hopper dredges are sometimes used to perform this type of dredging where certain conditions exist, making the overboard discharge of essentially all pumpings advantageous. Since 1957, private interests have developed self-propelled seagoing dredges of a type similar to Corps hopper dredges, but designed principally as more effective, high-capacity agitation dredging tools. To date, these dredges have been used only in Mexico and Venezuela, mainly in the latter country.

D. K. Ludwig, president of National Bulk Carriers, Inc., owners and operators of some of the world's largest oil tankers and ore carriers, reportedly conceived the idea of such dredges for dredging the bar at the mouth (called Boca Grande) of the Orinoco River that extends seaward over 25 miles. In 1956, this company converted a T-2 tanker to an experimental side-casting dredge. The conversion essentially consisted of installing two steam-turbine driven 36-inch suction, 32-inch discharge dredge pumps, with associated trailing suction dragarm assemblages of conventional hopper dredge design. However, no hoppers were provided, and the discharges from the two pumps are carried through individual 25-inch diameter overboard discharge pipes that form integral parts of a fixed (boom) structure extending 250 feet outboard of the starboard side of the vessel.

The material comprising the Boca Grande bar is a consolidated clayey silt with an in-place density averaging over 1,600 grams per liter. The controlling depth prior to commencement of work by the *Sealane* in December 1956 was approximately 12 ft. By May, 1961, the *Sealane* had removed over 100,000,000 cubic yards of this sediment through the entire length of the offshore bar, thereby providing a navigation channel for 60,000-d.w.t. ore carriers loaded to 33-ft. draft. Also, it is significant that since the method of operation involved resulted in gradual deepening over the entire length of the channel, interim use of the channel was permitted during the course of the work with attendant increasing benefits. The *Sealane's* success became apparent in its first year's operation. This prompted Venezuelan authorities to enter into a long-term charter agreement in 1958 with National Bulk Carriers for the construction of a specially designed 4-pump combination hopper and boom-type dredge and its use in deepening the Maracaibo Bar and the channel within Lake Maracaibo to permit navigation by the largest deep draft oil tankers.

This dredge named *Zulia* was built in Kure, Japan, in 1959 (figs. 10 and 11). It is a steam turbine vessel 548 feet long, 95 feet in beam (molded), and 40 feet deep, with 8,500-cubic-yard capacity hoppers and four 32-inch dredge pumps, each fitted with a suction dragarm assemblage (2 outboard or side drags and 2 inboard in ladder wells). The discharge piping is arranged so that the hoppers can be loaded in the conventional manner or the spoil can be pumped directly overboard through a single 57-inch boom-supported discharge line extending 328 feet from the ship's side. The boom can be rotated 180° from one side to the other, to permit discharge on either side. The *Zulia* undoubtedly is the largest and most productive earth-moving machine afloat. In its first year's operation it pumped about 57,000,000 cubic yards, mostly from the bar channel. The dredgings were thrown to either side of the channel, and reportedly most of the material was carried away by littoral currents and did not return to the channel.

Another dredge similar to the *Zulia* in size and pumping capability, except that it is diesel-powered and has a hopper capacity of 3,500 cubic yards, was built in 1961 by National Bulk Carriers. This dredge named *Icoa* was also constructed in Kure, Japan, for long-term charter by the Orinoco Mining Co., a subsidiary of U.S. Steel Corp., and is presently working in the Orinoco River to deepen further and maintain both the Boca Grande bar and river sections of the channel.

The Corps of Engineers has followed the development of the *Sealane*, *Zulia*, and *Icoa* with considerable interest. It is recognized that under certain conditions that the side-casting method of agitation dredging provides a practical means of removing large quantities of material relatively inexpensively and rapidly. In addition, when controlling depths do not permit conventional loading of hopper dredges during initial stages of work, side-casting may be employed economically and expediently to obtain an interim improvement in channel depths. Such conditions were encountered by the Corps in 1959 when it became necessary to dredge the entrance channel to the small harbor of Ontonagon, Mich., on Lake Superior. A very uniform controlling depth of between 9 and 10 feet ex-

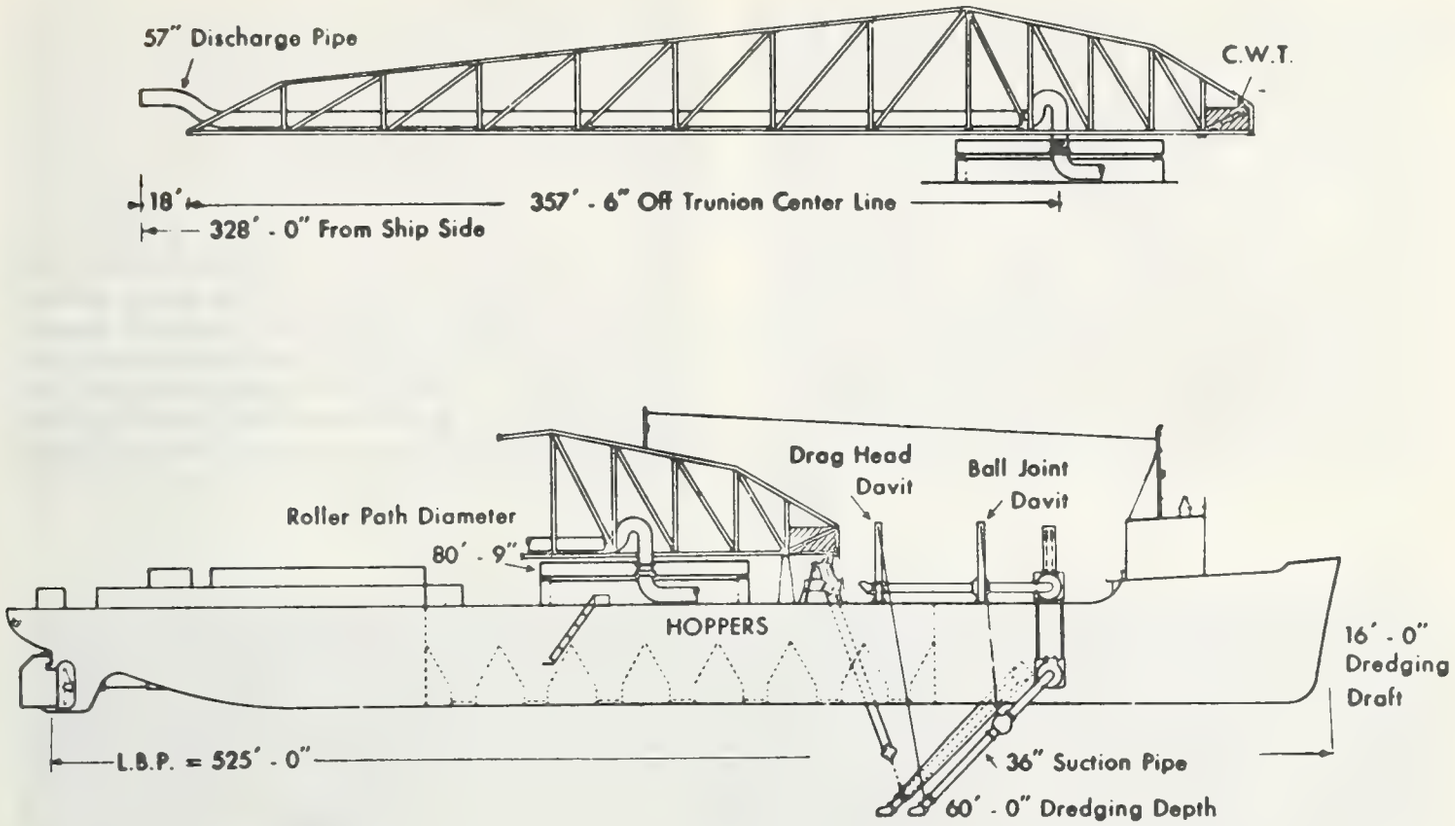


FIGURE 10. — Dredge *Zulia:* Elevations showing arrangement of boom discharge pipe.

FIGURE 11. — Dredge *Zulia* during side-casting operations.

isted in the channel section between pier breakwaters, which owing to draft requirements, precluded conventional use of the hopper dredge *Hains*. Therefore, this dredge was fitted with a discharge pipe extending approximately 20 feet from the ship's side to permit the dredging of a pilot channel by discharging directly overboard. The outer 10-foot section of pipe was elevated at a 20° angle, and at full pump power the point of discharge impact was approximately 45 feet from the dredge. By using this improvised setup, the critical part of the channel was deepened to 14 feet — sufficient for flotation required for normal loading and hauling operations. The material dredged was mostly a uniform fine sand with an in-place density of approximately 2,000 grams per liter. It settled rapidly despite prevailing current velocities of about 3 miles per hour, and after dredging soundings showed most of the side-boomed material remained where deposited. After this first experience with side-casting in the Great Lakes, the *Hains* has been equipped with a discharge pipe of 70-foot outreach (with impact about 100 feet off the ship's side) as shown in figure 12 and presently it is being used in selected projects in the Detroit district, particularly where the material may be deposited behind breakwater piers. Production in such cases is reportedly from 60 to 160 percent greater than when hopper dredging is done conventionally and the spoil is hauled a mile or more to a bottom dump.

FIGURE 12. — Hopper dredge *Hains* during side-casting operations at White Lake Harbor, Mich.

Side-casting capability also is being incorporated in the design of the proposed new dredge *McFarland*. This will be the first Corps of Engineers hopper dredge equipped with a revolving boom-supported discharge line for this purpose. The proposed vessel will be 300 feet long and will accommodate a fabricated aluminum boom 174 feet, 2 inches long (measured from the center of rotation to the discharge end), which will extend 138 feet, 2 inches beyond the nearest side of the dredge in the 90° working position. Side-casting dredging operations with this dredge in selected offshore channel sections of Gulf coast projects where prevailing cross-channel littoral currents occur are expected to reduce dredging costs. These projects include the Sabine-Neches outer bar, Freeport Harbor, Galveston entrance channel, Port Aransas, Brazos Island harbor, and Matagorda ship channel.

In addition, the Corps of Engineers is presently planning to convert two small surplus Navy vessels (one YSD and one YF) for exclusive use (no hoppers will be provided) as side-casting dredges in various shallow coastal inlet projects along the North Carolina coast and in similar exposed channels along the Louisiana coast, respectively.

Summary

In most hopper dredging work, spoil disposal is dumped in deep water. However, other methods of disposal listed have been or are being used under certain conditions:

(1) Dumping in rehandling basins (for subsequent disposal ashore by pipeline dredges);
(2) Agitation dredging;
(3) Sump rehandling (self-unloading);
(4) Direct pump ashore (self-unloading);
(5) Side-casting (boom discharge).

It is noteworthy that the two principal unorthodox hopper dredge disposal methods being given increasing attention involve widely different basic philosophies. On the one hand, the development of direct pumpout emphasizes the permanent removal of sediment so that it can never become a source of further shoaling or have any other detrimental effect on water resources development. At the other extreme, side-casting involves returning all the dredged material to the waterway directly adjacent to the shoal. This contrast becomes sharper when it is noted that the newest hopper dredges are equipped to dredge both ways; that is, they can pump directly ashore or side-cast, in addition to being able to operate conventionally by filling hoppers and dumping through bottom gates.

This increase in versatility of the hopper dredge manifests that there is no single solution to fit all dredging problems. Studies of harbors and estuaries are required to provide for an intelligent selection of spoil disposal techniques. These studies must include sedimentation and current analyses. From such analyses it can be concluded if the low cost and high-volume side-casting operation is advantageous, or if the slower and more expensive direct pumpout method should be utilized for the greatest long-range economy, or if the intermediate more conventional techniques are more appropriate.

The significant factor is that the modern hopper dredge is being designed and constructed to suit a wide range of requirements, and the task for the engineer is becoming one of giving greater emphasis to a truly considered discrimination of the various dredging and disposal methods that are available to him.

SOME ENGINEERING ASPECTS OF DISPOSAL OF SEDIMENTS DREDGED FROM BALTIMORE HARBOR

[Paper No. 66]

By MICHAEL A. KOLESSAR, *chief, Navigation Reports Section, U.S. Army Engineer District, Baltimore*

Synopsis

Disposal of sediments dredged from channels and harbors becomes increasingly difficult as waterfront property is developed commercially and disposal areas become scarce. This paper presents a method for obtaining more effective use of a limited area. It describes laboratory tests made to determine the bulking characteristics of the sediments when placed in a confined area by hydraulic pipeline dredge.

It has been said that great ports are usually geographic accidents. They achieve their early prominence by virtue of their location with respect to centers of population and overland transportation networks. Interestingly, however, the further development of the port quite often depends, to a large degree, on the geological history of the region. The ease with which channel dimensions can be increased and maintained is closely related to the geomorphology of the area and particularly to sedimentation processes that for centuries have contributed to the makeup of the river bottom.

The port of Baltimore is a striking example of the strong influence these processes have on

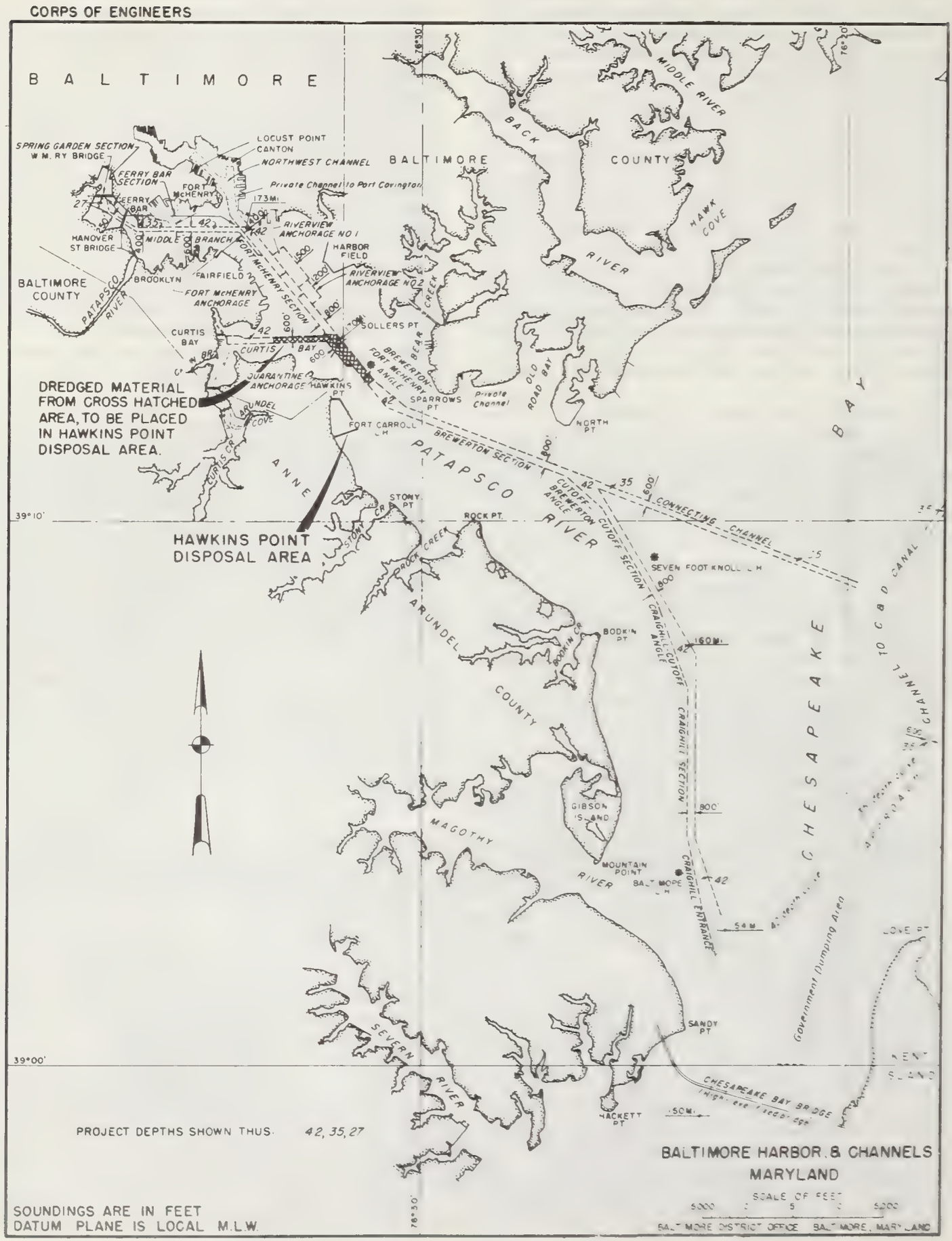


FIGURE 1. — Upper reaches of Baltimore Harbor and channels, Maryland.

the development of channels for deep-draft traffic (fig. 1). Baltimore is a fall-line city, being on the boundary between the Atlantic Coastal Plain and the Piedmont Plateau, and is located at the head of the navigable part of the Patapsco River, which is a tidal estuary on the west side of Chesapeake Bay. Chesapeake Bay, actually a large tidal estuary, represents an ancient drainage system, which was drowned toward the end of the Pliocene epoch. Further modification was introduced by the rising and falling sea level upon a presumable stable land surface during the following Pleistocene epoch.

The valleys of the ancient drainage system in unconsolidated Coastal Plain deposits of pre-Pleistocene Age were subsequently filled by alluvium eroded from highlands and by deposits of Recent Age after inundation by marine waters. It is in these recent deposits of silt, clay, and sand that the navigation channels in the Patapsco River are dredged and maintained. The existence of these soft sediments has contributed largely to the growth of the Baltimore port. The material is relatively easy to dredge, and a minimum of maintenance is required because the shoaling rate is quite small. Another advantage is that the soft bottom of the channel presents far less danger of damage to heavily laden vessels operating with little clearance under the keel. The presence of these fine-grained materials, which in the past have made it feasible to keep pace with the ever-increasing demands for channels to accommodate larger and larger carriers, now presents one of the most difficult problems in further development of the port.

The problem is one of disposal of the dredged material. Waterfront property on the periphery of the harbor is highly industrialized and valued, in some cases, as high as $15,000 per acre. Large land areas for dumping are simply not available within economic distances from the channels to be dredged. Water areas adjacent to the shoreline are not often usable because depths of water and poor foundation conditions make the cost of necessary retaining dikes prohibitive. The dredged material, composed of silt and clay with a dry weight of about 35 pounds per cubic foot, is not ideal fill, and property owners are not eager to obtain such material for enhancement of their lands. In view of these circumstances, the efficient use of the limited areas available for disposal of spoil is not only desirable but becomes a very important factor in determining the method of dredging that will be used.

This paper presents an approach to the design of an optimum-size disposal area to be used for disposal of material dredged by a hydraulic pipeline dredge. The author, who was one of the engineers who formulated the plan, will readily admit that methods used may appear cumbersome to those more familiar with sedimentation processes. It is his hope, however, that if any fundamental concept has been missed or improperly applied, informed readers will bring the matter to his attention.

One of the disposal areas made available by local interests as part of the local cooperation required by the authorizing Act for the Baltimore Harbor project consisted of about 130 acres of water area immediately channelward and adjacent to lands owned by two different corporations. One of the riparian owners, in giving permission to use the offshore area, specified that the final elevation of the fill would not exceed 9 feet above mean low water. The other riparian owner requested that fill be not less than 4 feet above mean low water and not higher than adjacent fast lands that have an elevation of about 35 feet. The average depth of water in the area was about 6 feet but a submarine bar roughly parallel to the shore line and about 2,000 feet therefrom rose to within 3 feet of the water surface. Subsurface explorations confirmed the belief that the bar was composed of hard material and would provide adequate foundation for a moderately high dike. The borings also showed that, within the disposal area, there were pockets of sand and other granular materials which could be used for construction of a dike.

A rule of thumb in general use in the dredging industry for determining size of a disposal area needed for a continuous hydraulic dredging operation is that the volume of the enclosure should be a minimum of 2 to 3 times the in situ volume of material to be removed from the cut.

In this particular instance, however, it was believed that considerable refinement was necessary to assure that the final elevation of the fill would be within the limits specified without having to resort to intermittent work stoppages while the fill decanted. Information was also needed on the retention time that would be required in the disposal area to assure that the effluent over sluiceways would not exceed by more than the specified 20 grams per liter the weight of ambient water in the channel. It was decided to simulate the mixture that might be expected in the dredge pipe and observe the rate of sedimentation in transparent vertical stand pipes.

Five transparent plastic tubes of about 4 inches in diameter were erected in a freight elevator shaft at the Baltimore District Office of the Corps of Engineers. Two of the tubes were slightly over 6 feet in length, two were a few inches longer than 12 feet, and one was

about 24 feet long. The 12-foot lengths corresponded to the probable protoype dimensions of the column of mixture that would occur in the disposal area. The 24-foot tube was erected to simulate the placing of material in a disposal area containing 6 feet of water.

Earlier observations of various mixtures in 1,000 cc. graduates showed, as might be expected, that the rate the interface between clear water and the turbid mixture lowered in the graduate varied in proportion to the amount of mixing water. But since there was some question as to whether the consolidated characteristics of the sediment in the small test could be correlated with the higher columns in the disposal area, it was decided to make direct observation of a full-size column under conditions more nearly resembling the actual operation.

It was not possible to find general agreement among dredging contractors regarding the density of material that would be transported through the proposed 2-mile long dredge pipeline. Most agreed that about 25 percent of the mixture in the pipeline would be solids, but they were not certain if the measurement was by weight or volume.

Laboratory analysis of 14 samples taken in the area to be dredged showed that the in situ density of the material as measured by a Bico-Braun Mudwate hydrometer averaged about 1,300 grams per liter (table 1). Specific gravity

TABLE 1.—*Soils data for Curtis Bay channel obtained November 1960*

[Weight of water=62.355 lb. at 60 F°.]

[Water content, $w\% = \frac{\text{weight of water}}{\text{weight of solids}} \times 100$]

Hole No.	Sample No.	Depth below water surface	Water content w%	In situ density		Specific gravity of solids	Void ratio	Grain size	
				G./liter	Lb./cu. ft.			D_{50}	D_{10}
		Feet	*Pct.*						
C–1	2	30–35	193.3	1,258	78.4	2.728	5.856		
C–1	3	35–40	164.6	1,285	80.1		5.075	0.0014	0.00015
C–1	4	40–45	150.5	1,319	82.2	2.696	4.413		
C–2	1	20–25	189.0	1,274	79.4		5.693		
C–2	2	25–30	199.9	1,260	73.6	2.794	6.063	.0047	.00040
C–2	3	30–35	158.1	1,313	81.9		4.585		
C–2	4	35–40	161.9	1,311	81.7		4.622	.0025	.00025
C–2	5	40–45	158.8	1,319	82.2	2.716	4.478		
C–3	1	23–28	111.0	1,397	87.1		3.413	.0020	.00010
C–3	2	29–30	25.3			2.693		.28	.0090
C–4	1	25–30	144.4	1,337	83.4	2.683	4.079		
C–4	2	30–35	159.5	1,317	82.1		4.406		
C–4	3	35–40	169.2	1,295	80.7		4.934	.0014	.00025
C–4	4	40–45	165.0	1,309	81.6		4.705	.0011	.00030

of solids ranged from 2.70 to 2.79 and void ratio ranged from 3.413 to 6.063. Mechanical analysis showed that the median diameter of grains was about 0.0022 mm.

It was decided to place mixtures of 1 part of 1,300-gram-per-liter material and 4 parts of water by volume in one of the 6-foot tubes and in one of the 12-foot tubes. To obtain an approximation of the effect of a denser mixture, mixtures of 1 part 1,300-gram-per-liter material and 1 part of water by volume were placed in the other 6- and 12-foot tubes. Mixing water was water taken from the site of the proposed dredging and had a specific gravity of 1.003. These proportions were selected as representing the probable upper and lower limits of the density of material in the dredge pipe line.

The 24-foot tube was charged with 6 feet of brackish water and 6 feet of 1-to-1 mixture was added. After 4 days the supernatant liquid was drawn off and another 6-foot lift of 1-to-1 mixture was added. The process was repeated on the 27th day. The elevation of the interfaces was recorded and plotted against elapsed time. The results of the tests are shown on figures 2 and 3. Numbers along the curves represent the relation of the volume of the mixture containing sediments in suspension as compared to the original in situ volume of material of 1,300 grams-per-liter density contained in the mixture. This relation is referred to later in this paper as the bulking factor.

In computing the volume required in the disposal area, the following assumptions were made:

1. Dredging would be by hydraulic dredge with 27-inch discharge that would deposit about 6,350 cubic yards of mixture in the disposal area per hour.

2. Pumping would be in progress 22 hours per day for 25 days per month.

3. Discharge over spillways would not exceed 3 inches in depth, and density of effluent over spillways would not exceed 1,020 grams per liter.

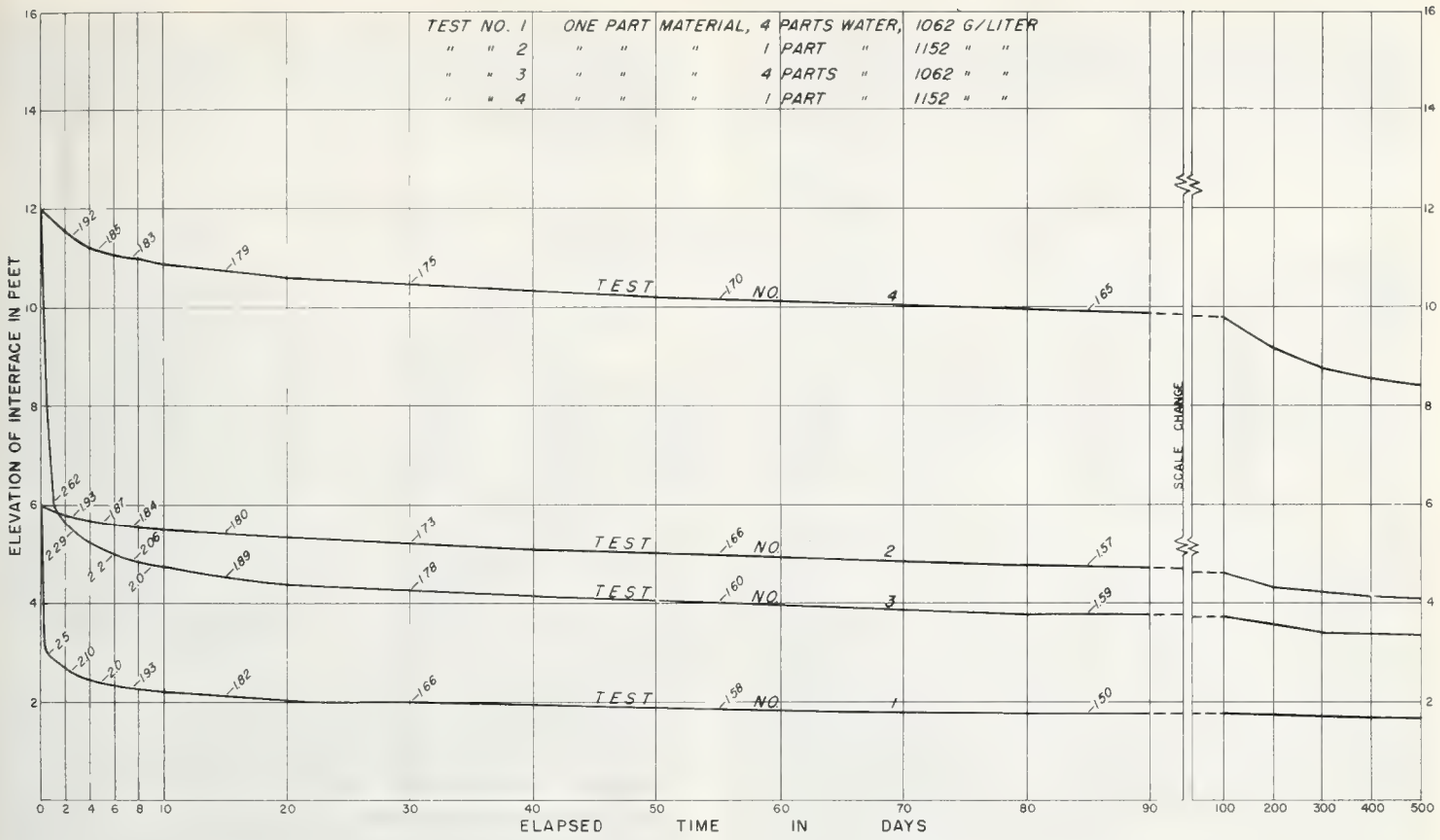

TEST NO. 1 ONE PART MATERIAL, 4 PARTS WATER, 1062 G/LITER
" " 2 " " " 1 PART " 1152 " "
" " 3 " " " 4 PARTS " 1062 " "
" " 4 " " " 1 PART " 1152 " "
TEST NO. 4
TEST NO. 2
TEST NO. 3
TEST NO. 1
SCALE CHANGE
ELEVATION OF INTERFACE IN FEET
ELAPSED TIME IN DAYS

FIGURE 2. — Relationship between elevation of interface with elapsed time: Tests 1 through 4.

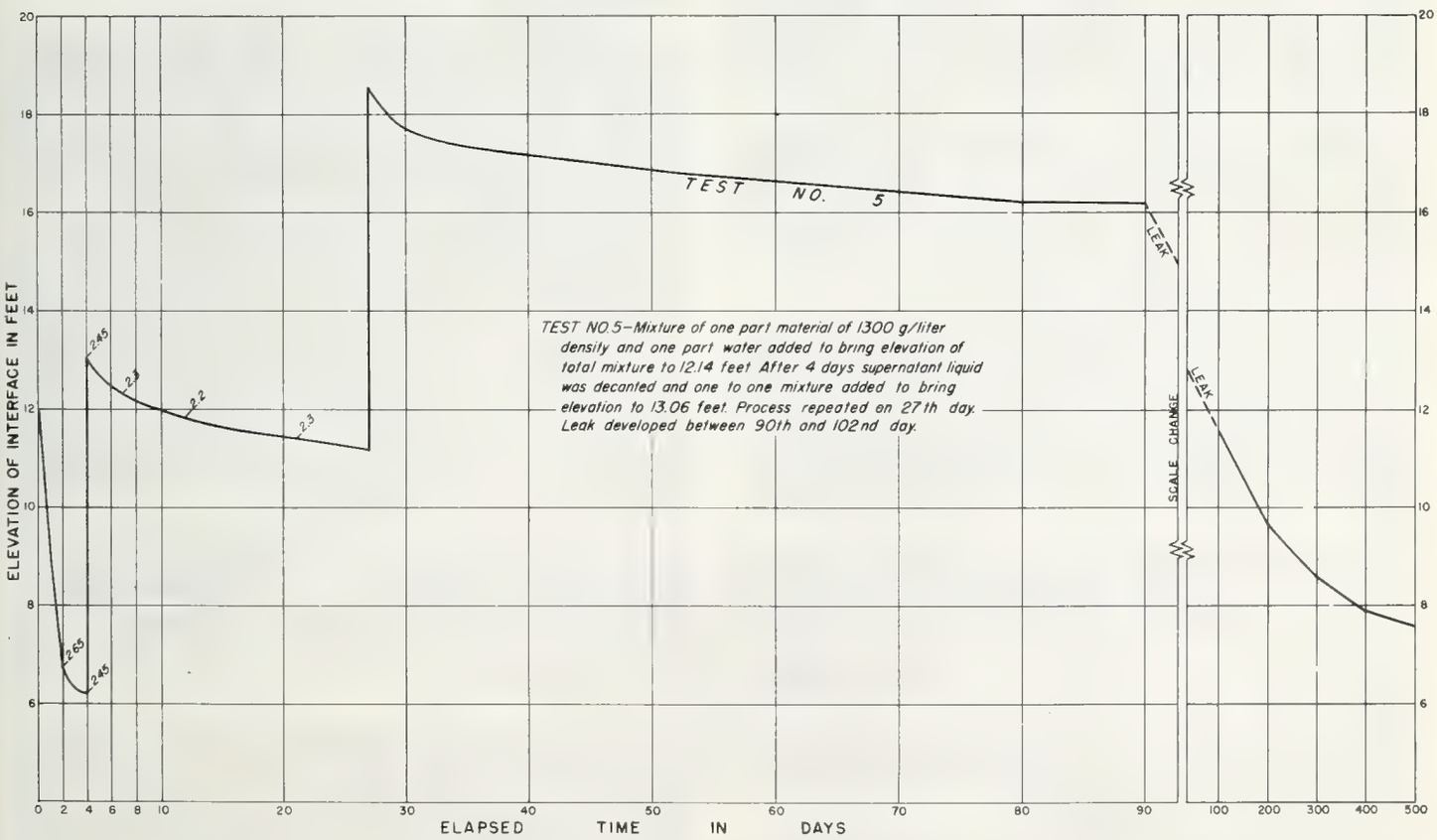

TEST NO. 5
LEAK
LEAK
TEST NO.5—Mixture of one part material of 1300 g/liter density and one part water added to bring elevation of total mixture to 12.14 feet. After 4 days supernatant liquid was decanted and one to one mixture added to bring elevation to 13.06 feet. Process repeated on 27th day. Leak developed between 90th and 102nd day.
SCALE CHANGE
ELEVATION OF INTERFACE IN FEET
ELAPSED TIME IN DAYS

FIGURE 3. — Relationship between elevation of interface with elapsed time: Test 5.

4. Losses over the spillway would be approximately equal to excess overdepth dredging.

5. The density of material in the discharge pipe would be between 1,060 and 1,152 grams per liter.

6. A maximum of 1 million cubic yards in place in the channel would be dredged and placed in the disposal area during 1 month, and the entire pumping operation would be complete in 100 days.

TABLE 2.—*Computed volume of fill after 100 days of pumping*

(In thousands of cubic yards)

Portion of job (percent)	Volume cut	Days in disposal	Bulking factor test[1]				Volume of fill test			
			1	2	3	4	1	2	3	4
30	960	70–100	1.50	1.57	1.59	1.65	1,440	1,507	1,526	1,584
30	960	40–70	1.58	1.66	1.60	1.70	1,517	1,594	1,536	1,632
20	640	20–40	1.66	1.73	1.78	1.75	1,062	1,107	1,140	1,120
10	320	10–20	1.82	1.80	1.89	1.79	582	576	605	573
5	160	5–10	1.93	1.84	2.06	1.83	309	294	330	293
5	160	0–5	2.10	1.93	2.29	1.92	336	309	367	307
100	3,200	.....	.....	.....	.....	.....	5,246	5,187	5,504	5,509
Ratio of volume in fill to volume in cut							1.63	1.62	1.72	1.72

[1] Test No. 1 was 6-foot tube of 1 to 4 mix (1,062 g./l.), test No. 2 was 6-foot tube of 1 to 1 mix (1,152 g./l.), test No. 3 was 12-foot tube of 1 to 4 mix (1,062 g./l.), test No. 4 was 12-foot tube of 1 to 1 mix (1,152 g./l.).

Table 2 presents a summary of the computation of the volume required in the disposal area to accommodate 3.2 million cubic yards of material to be removed from the cut. Bulking factors as determined by tests 1 through 4 are shown for increments of time varying from 5 days to 30 days, the shorter times being selected for the periods when the curves showed the greater rates of change. The totals at the bottom of each of the last 4 columns represent the maximum volume the fill will attain and the volume that must be made available in the disposal area.

The density of the initial mix had no appreciable effect on the volume of the fill at the end of the pumping time. Also, the volume of fill as determined by the 12-foot tests indicated a higher relationship of fill to initial volume than did the tests using 6-foot tubes.

The estimated volume of the fill computed on the basis of the test data is believed to be somewhat higher than would actually occur because the test did not take into account the probability that a substantial part of the material dredged would be transported to the disposal area in the form of balls and would not go into suspension. It was considered safe to assume, therefore, that the maximum volume of fill would be not more than about 70 percent greater than the volume in the cut. This narrowing down of the range of bulking that could be expected made it possible to make a better determination of the economical height of the dike and the optimum amount of material to be placed in the disposal area.

The disposal area at Hawkins Point was constructed in September 1962. The contractor pumped up dikes to about mean low water, using borrow areas within the disposal area itself. By dragline he brought material from the inside slope to bring the top of the dike to about 12 feet above mean low water. By excavating material from within the disposal area for dikes, he increased the capacity below mean low water. A hydrographic survey was made after dikes were constructed but before filling operations began. Further surveys after the fill is complete will be made to compare the actual prototype conditions with those predicted by the laboratory tests. From initial observations, close agreement is expected.

CHANNEL SEDIMENTATION AND DREDGING PROBLEMS, MISSISSIPPI RIVER AND LOUISIANA GULF COAST ACCESS CHANNELS

[Paper 67]

By AUSTIN E. SMITH, *Chief, Navigation and Dredging Branch, U.S. Army Engineer Division, Lower Mississippi Valley and Mississippi River Commission, Vicksburg, Miss.*

Synopsis

Sediment has been a major navigation problem at the Mississippi River Passes since Sieur de la Tour completed the plans for New Orleans. Channel shoaling in the Mississippi River between Cairo, Ill., and New Orleans, La., has

been a major navigation problem since 1870. The solution to the Passes problem, where the salt-water wedge and the Mississippi River sediment meet, is being resolved by a combination of jetties, contraction works, and dredging. The annual channel shoaling on the 200 crossings between Cairo, Ill., and Baton Rouge, La., has been battled by the dustpan dredge since the development of this unique equipment by the Mississippi River Commission in 1895. Today this problem is being combated by channel stabilization works and dredging.

Silting in the Atchafalaya River, including the basin, the lake section, and the Gulf access channel below Morgan City, is being attacked by a major dredging program. Channel silting in the Calcasieu River 40-foot ship channel is being met by new construction dredging, including retention dikes, enlargement of the existing waterway, and maintenance dredging. Channel sedimentation on the new 36- by 500-foot gulf outlet channel is expected to be heavy. Experience on similar channels and that gained during current construction indicates that channel siltation in this new tidewater channel to New Orleans will be heavy from wave wash of passing vessels and sea action, particularly in the seabar section where the littoral drift is athwart the entrance channel.

Channel Sedimentation — Historic Problem

During 27 to 30 May 1962, 28 motor vessels with 272 barges were delayed a combined total of 831 hours and 25 minutes due to the loss of project depth in Grand Lake Crossing (mile 504) on the Mississippi River. Until dredged, the controlling depth in this river crossing was 7 feet, whereas the drafts of the loaded barges and towing vessels ranged from 8 to 10 feet. Again, on 15, 21, and 22 August 1962 the navigation channel was blocked by sediment at three other separate shoaled crossings. These channel blocks occurred as the Mississippi River fell rapidly from middling stages and before the Corps of Engineers dredge plant could deepen a number of heavily shoaled crossings. These 1962 low-water navigation difficulties attest to the fact that sediment in the channel remains the major problem in providing a dependable navigation channel on the Lower Mississippi River, the congressionally assigned task of the Mississippi River Commission, Corps of Engineers, U.S. Army.

River engineers have been concerned with Mississippi River sediment since Sieur de la Tour completed the plan of New Orleans and Sieur de Paugér constructed the post called Balize (the buoy) at the mouth of Southeast Pass (Pass a l'Outre), circa 1724, to serve as a guide to ships wishing to make passage over the 10-foot-deep seabars and ascend the river. They tried various schemes to combat the silt and deepen the selected Passes, but it remained for Eads and Corthell in 1875 to design and construct the first successful channel control works which, with dredging, provided a deep-draft channel via South Pass.

In his 1821 examination of the Mississippi River, Captain H. Young of the U.S. Topographic Engineers found the navigation channel infested with logs, snags, and sawyers deeply embedded in Mississippi River silt. Captain Henry M. Shreve removed the snags, but it remained for the Mississippi River Commission to develop, in 1895, the unique dustpan dredge, which has provided a dependable method of maintaining the project navigation channel by removing the annual high-water siltation from those crossing channels that do not erode as the river falls to low stages.

On 24 July 1862, during the War Between the States, Adm. David G. Farragut moved his Union fleet of war vessels from siege before Vicksburg, Miss., downstream to the deeper waters in the Baton Rouge-New Orleans sector to prevent these overdraft vessels from being trapped in hostile territory behind shallow crossings as the river fell to low stages.

By 1870 larger and deeper draft steamboats were being constructed for economy of transportation. During years of extreme low water, overdraft steamboats were forced to lay by for 1 to 3 months of the year because of shoaled crossings. This critical situation led the U.S. Congress to direct that extensive studies be made by the Army Corps of Engineers of the Mississippi River navigation problems and to establish the Mississippi River Commission in 1879.

This Commission gave immediate consideration to the channel siltation problem. It made extensive studies and examined all phases of the Mississippi River regimen, including the movement of sand waves on the riverbed and other siltation phenomena associated with river meander, such as bank caving and the enlargement of the meander loops by alluvial deposits. First attempts to cope with the problem were: (1) regulation of the low-water channel by the construction of permeable dikes, and (2) closure of secondary channels at Plum Point, Tenn., and Lake Providence, La. The dikes and closure dams, as then constructed, were inadequate to withstand the forces of the river. After exhaustive experiments, the Commission adopted dredging as the positive method of maintaining a dependable navigation channel against the cyclic siltation in the Mississippi River. In combating the silt problem by dredging, the

Commission was fortunate in the early development of the dustpan-type dredge, which remains the optimum plant for dredging Mississippi River crossings.

Mississippi River Channel Shoaling Problem — Cairo to Baton Rouge

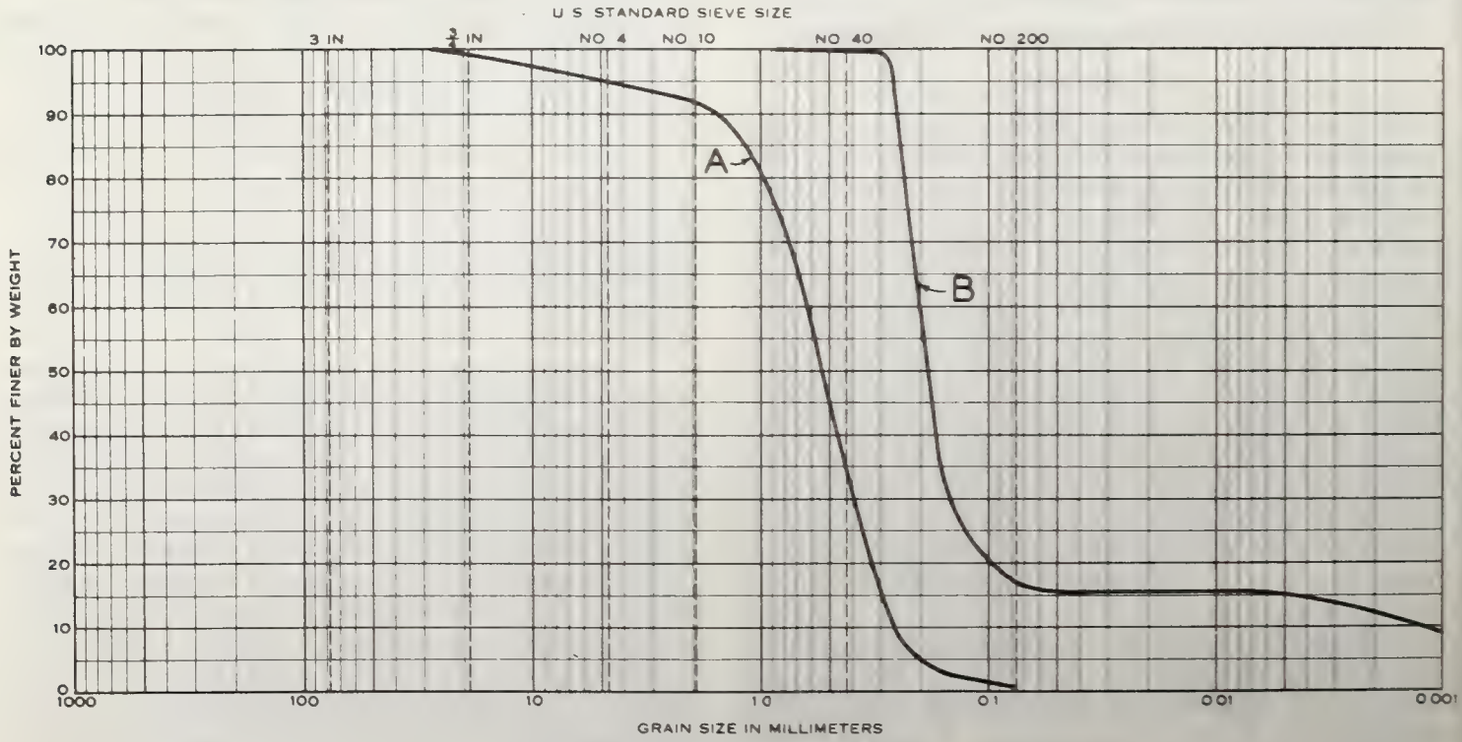
In the analysis of the Mississippi River dredging problems, sediment is considered to be any material (silt, sand, or clay) that is deposited in the channel and has to be removed by dredging. Figure 1 shows the mechanical analysis of a sample taken from the bed of the Mississippi River at a point 813 miles above the Head of Passes and of a sample taken in Southwest Pass near the mouth of the Mississippi River.

The two major maintenance requirements are (1) removal of high-water shoaling from the Mississippi River crossings, and (2) removal of highwater shoaling in the "shabby" reaches where the river's flow is bifurcated and trifurcated. Figure 2 shows a typical Mississippi River meander and the crossings that usually shoal at high water and require dredging at low stages.

Bank caving has been a major source of the material that is deposited in river crossings and "shabby" reaches during high-river stage. In his 1892 study and analysis of the 1877-83 and the 1892 hydrographic and caving bank surveys of the Mississippi River, J. A. Ockerson, member, Mississippi River Commission, found that the total length of caving banks between Cairo, Ill., and Donaldsonville, La., was then 921 miles, and that the average annual volume of caving per mile from Cairo to Donaldsonville was then 1,003,579 cubic yards.

From 1945 prototype studies of Mississippi River bank caving, occurring between the 1931 and 1941 hydrographic surveys, Austin Smith and Edgar McCleave, staff engineers of the Mississippi River Commission, reported as follows:

> Without stabilization there will be continued caving of the bank. The amount of annual bank caving on the Mississippi River between Cairo, Illinois, and the mouth of Old River near Angola, Louisiana, a river distance of approximately 770 miles, is some 800,000,000 cubic yards. While the magnitude of this bank caving, when indicated only in figures, might not seem excessive, it would require about seventy-five 30-inch dredges working constantly to move this volume of material annually. The Mississippi River is the world's biggest dredge.



Sample A obtained from bed of Mississippi River 813 miles above Head of Passes.
Sample B obtained from Southwest Pass near the mouth of Mississippi River.

FIGURE 1. — Grain-size distribution at Mississippi River crossing below Cairo and at mouth of Mississippi River.

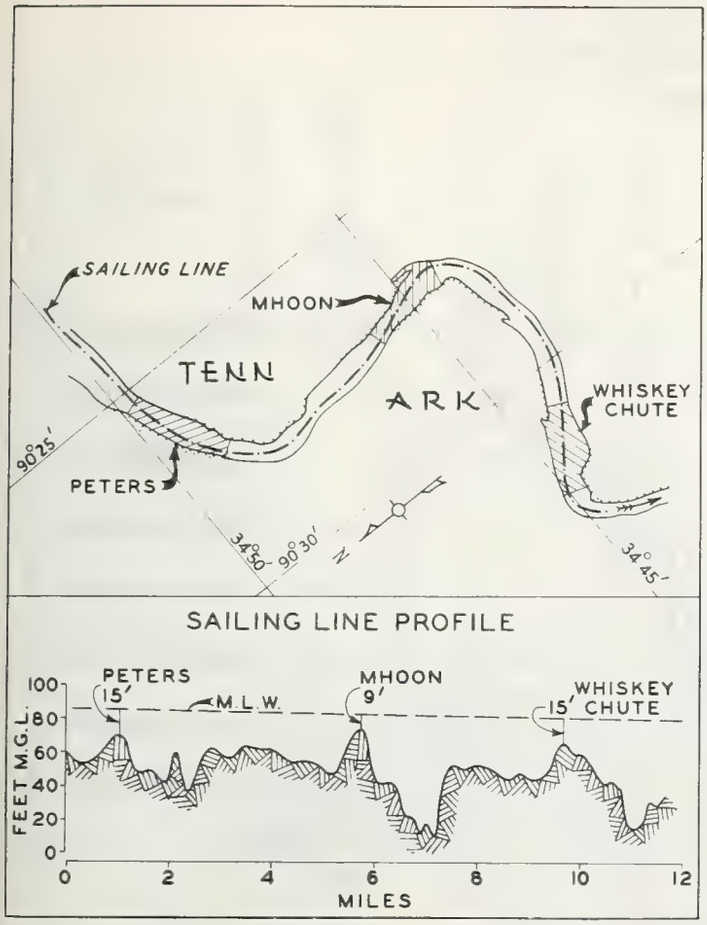


FIGURE 2. — Typical crossing.

Figure 3 shows the 1931-41 caving bank history between Cairo and Old River, as described above.

These two caving bank studies, some 50 years apart, show that in its natural regimen annual bank caving on the Mississippi River between Cairo and Donaldsonville averaged over 1,000,000 cubic yards per mile. Both reports show that the heaviest bank caving occurred between the mouth of the Arkansas River and Natchez, Miss. The influence of this source of channel sediment on the Mississippi River regimen is marked.

The Mississippi River Commission in its study and review report on the Mississippi River between Cairo and Baton Rouge, dated 14 February 1944, House Document No. 509 (78th Congress, 2d Session), noted that the source of sediment from caving banks would be markedly reduced when the river is stabilized, and stated:

> Stabilization of the banks will reduce the sand in motion, will fix the location of crossings and reduce the amount of dredging required to maintain the 9-foot channel of the existing project. It is estimated that the existing authorization for maintenance dredging will therefore be ample to secure a minimum low-water depth of 12 feet after stabilization is accomplished.

In concluding its report the Commission stated:

> The Commission is of the opinion that stabilization of the river is necessary to retain reduction in floods obtained by channel rectification and is advisable for the purpose of safeguarding the main Mississippi River levees and protecting the investment which they represent. The Commission is of the further opinion that such stabilization may materially increase the flood-carrying capacity of the river channel and, together with maintenance dredging already authorized, will provide a minimum depth of 12 feet at low water for navigation.

With the advance of the stabilization program there has been some reduction in the annual dredging requirements to maintain the interim 9- by 300-foot navigation channel.

Channel Crossing Shoaling — Cairo to Baton Rouge

Between Cairo and Baton Rouge there are some 200 river crossings, including a number of "shabby" reaches where there is divided flow. As the river falls to low stages, some 40 to 85 crossings do not scour and thus require annual dredging. The annual dredging estimates are based on the assumption that at least 60 crossings will require dredging during normal low-water seasons.

The number of crossings requiring dredging in the fiscal years 1953-61, inclusive, is shown on table 1. The number of crossings having a controlling depth of less than 12 feet in selected years between 1936 and 1960 is shown on table 2.

TABLE 1.—*Number of crossings dredged between Cairo, Ill., and Baton Rouge, La., 1953–62*

Fiscal year	Vicksburg lowest flow	Number of crossings dredged at—			Total
		Memphis district	Vicksburg district	New Orleans district	
	C.f.s.				
1953	132,000	60	14	1	75
1954	136,000	54	31	1	86
1955	145,000	37	16	.....	53
1956	128,000	35	29	.....	64
1957	133,000	33	23	.....	56
1958	230,000	30	14	.....	44
1959	198,000	33	15	.....	48
1960	162,000	33	11	.....	44
1961	205,000	35	9	.....	44
1962	246,000	37	5	.....	42

TABLE 2.—*Number of crossings with less than 12-foot depth, Cairo, Ill., to Baton Rouge, La., for selected years, 1936–60*

Year	Crossings with less than 12 feet	Vicksburg low flow
	Number	*C.f.s.*
1936	130	94,000
1938	71	157,000
1939	153	100,000
1942	41	254,000
1943	74	161,000
1956	77	128,000
1960	54	187,000

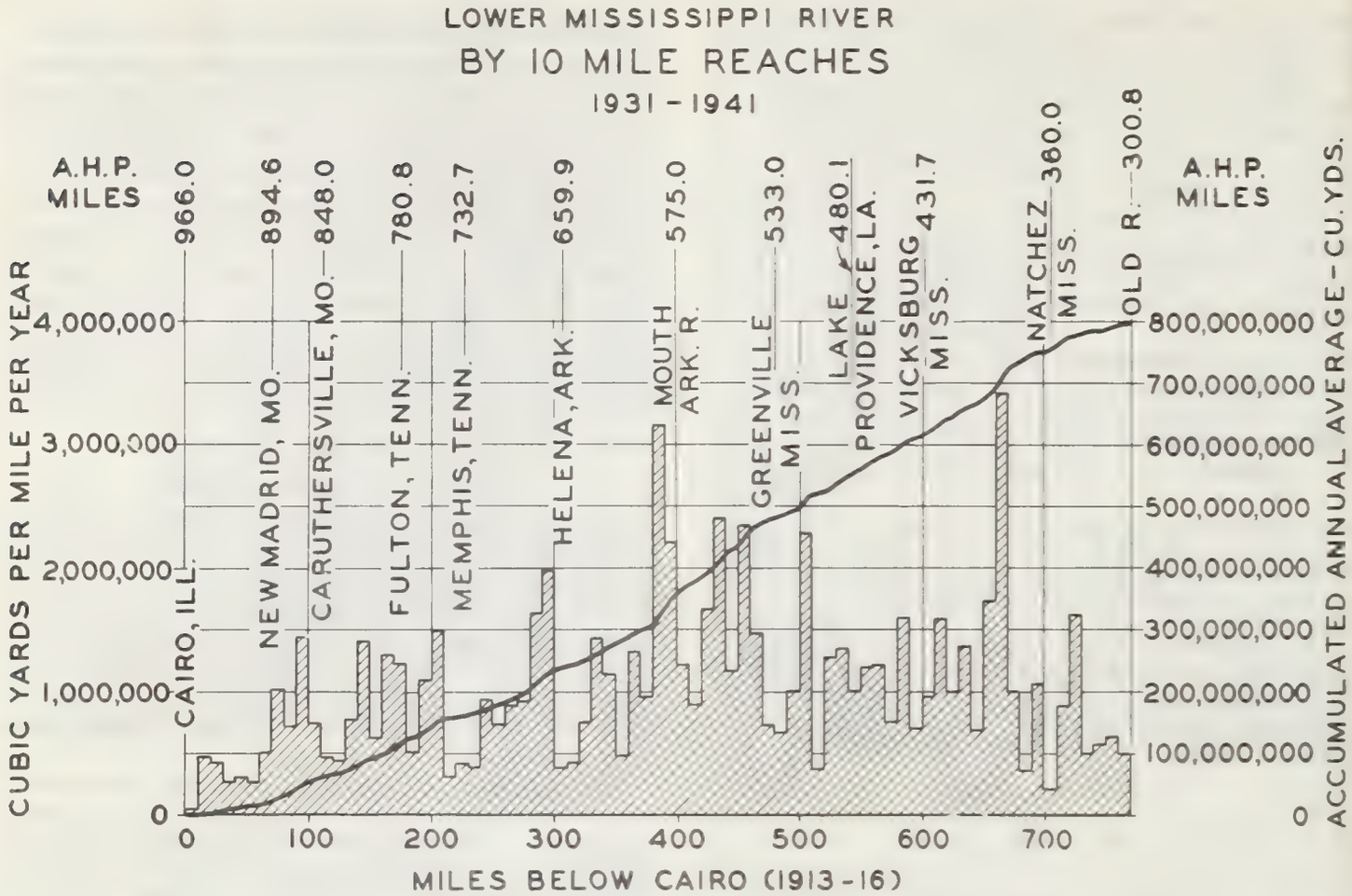


FIGURE 3. — Caving bank history.

Since the crossing bars and divided flow reaches build up during high-water stages and about one-third of them do not scour out satisfactorily of their own accord, steps must be taken at the beginning of each low-water season to (1) determine which crossings will require attention and mark the best channel available until improvements can be made, (2) make "before dredging" surveys for getting the dredging work properly laid out, (3) move dredges to the jobs and deepen the shallow crossings at the proper time, and (4) check all crossings regularly during the low-water season and dredge if necessary.

Annual Maintenance Dredging Requirements — Cairo to Baton Rouge

The annual maintenance dredging required to provide a dependable 9- by 300-foot channel from Cairo to Baton Rouge, a distance of approximately 737 miles, ranges from an extreme low of 20,000,000 to a high of about 45,000,000 cubic yards. The quantity of maintenance dredging required during the fiscal years 1930 through 1962 is shown on table 3. This large amount of maintenance dredging is accomplished annually by four large dustpan dredges, three of which are located in the Memphis district and one in the Vicksburg district. This dredging work is performed annually between May and January at an annual cost of about $4½ million. The schematic diagram of the operation of the dustpan dredge is depicted on figure 4.

The dustpan dredging requirements listed in table 3 show an increase during 1932-45, reflecting a channel rectification program that included 16 cutoffs and other major channel realinements. While channel stabilization accomplished since 1945 has had a salutary influence on the annual dredging requirements, there are other factors that must be considered in estimating the dredge capacity needed to maintain a dependable 12-foot channel between Cairo and Baton Rouge. These factors are (1) the increased vessel draft that is depicted on figure 5, (2) increase in lost dredging time due to passing navigation, ranging up to 16 hours in some situations, and (3) outsize tows that require greater than the project-width channel for safe passage during low river stages.

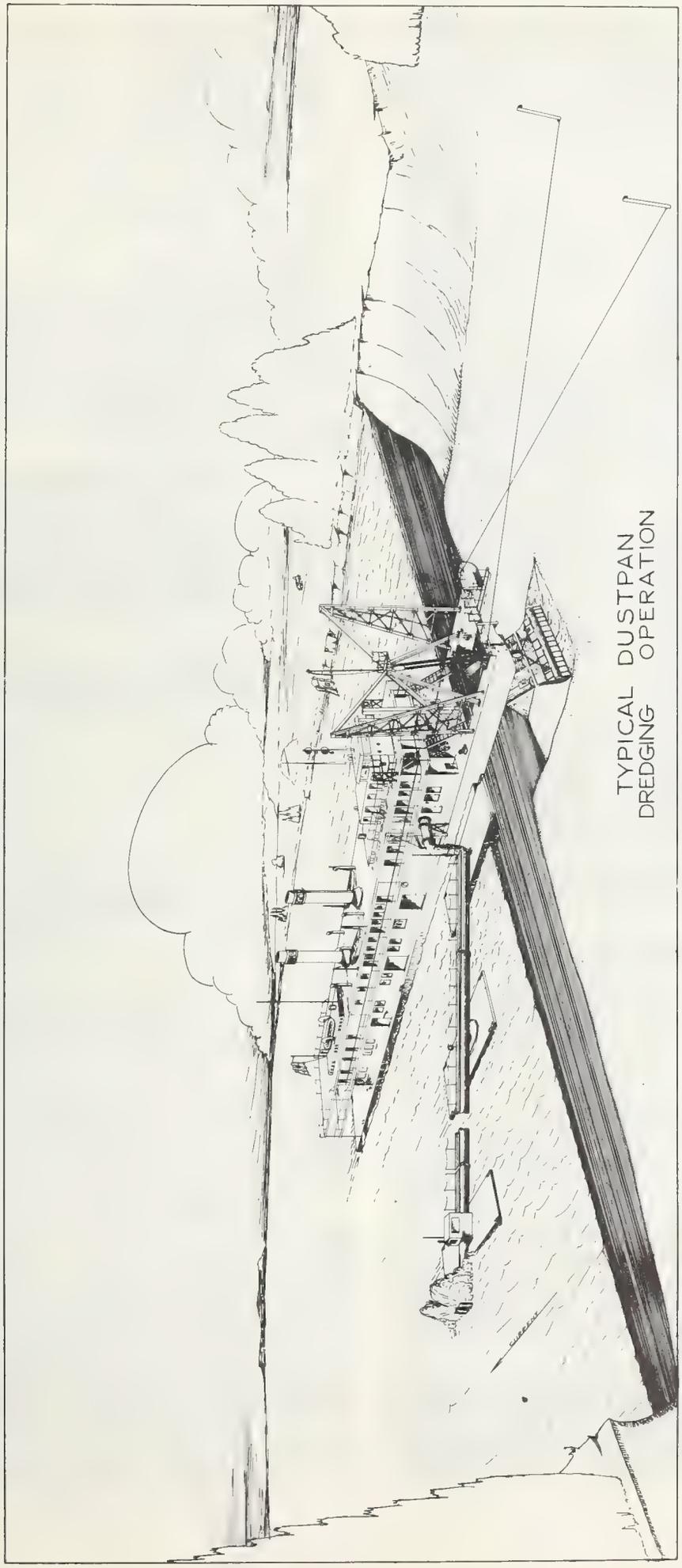


FIGURE 4.—Schematic diagram of dustpan dredge operation.

TABLE 3.—*Maintenance dredging between Cairo, Ill., and Baton Rouge, La., 1930–62*

Fiscal year	Quantity excavated in—			Total
	Memphis district	Vicksburg district	New Orleans district	
	Cu. yd.	*Cu. yd.*	*Cu. yd.*	*Cu. yd.*
1930	5,044,507	0	0	5,044,507
1931	7,579,131	0	0	7,579,131
1932	8,916,679	0	84,000	9,000,679
1933	12,127,300	184,363	258,653	12,570,316
1934	29,673,173	437,690	208,215	30,319,078
1935	27,554,524	183,310	97,300	27,835,134
1936	17,596,668	918,744	0	18,515,412
1937	31,069,025	2,625,898	0	33,694,923
1938	26,108,704	1,483,501	99,415	27,691,620
1939	19,624,409	3,105,420	0	22,729,829
1940	31,168,225	4,262,679	1,086,660	36,517,564
1941	25,814,861	6,201,059	[1] 8,116,124	40,132,044
1942	19,713,591	[2] 6,200,000	[1] 11,407,272	37,320,863
1943	15,466,388	[2] 6,150,000	[1] 13,930,593	35,546,981
1944	27,037,000	[2] 8,450,000	[1] 5,478,000	40,965,000
1945	25,162,000	7,686,000	[1] 9,174,000	42,022,000
1946	20,278,277	5,420,075	[1] 6,924,800	32,632,152
1947	17,143,575	10,373,460	551,775	28,068,810
1948	30,407,310	10,956,235	1,138,141	42,501,686
1949	27,901,570	9,324,614	1,762,142	38,988,326
1950	29,242,773	9,223,574	432,294	38,898,641
1951	25,987,195	6,991,611	0	32,978,806
1952	14,271,086	2,850,564	1,490,285	18,611,935
1953	35,163,231	5,358,374	132,545	40,367,307
1954	39,830,162	6,156,021	373,852	46,403,493
1955	29,839,459	7,288,930	0	37,128,389
1956	33,286,998	8,706,476	0	41,993,674
1957	34,390,837	5,806,922	0	34,390,837
1958	22,204,396	6,777,131	0	28,981,527
1959	26,928,000	7,230,000	0	34,158,000
1960	25,335,247	4,942,497	50,300	30,328,044
1961	24,709,000	4,943,000	218,000	29,870,000
1962	28,213,000	7,502,000	199,000	35,914,000

[1] Under channel conditions after major rectification of channel trace.

[2] Quantities are approximate and are adjusted to reflect only maintenance dredging work, excluding all improvement dredging.

Baton Rouge to the Head of Passes

In this sector of the Mississippi River, where a 35- by 500-foot channel is maintained, there are some six crossings that may require dredging, depending upon the magnitude and duration of the high water. Here, as the river falls to low stages, high-water shoaling is removed by dredging, presently with the use of a hopper dredge. Recent prototype experiments with a dustpan dredge have proved successful at crossings at Red Eye and Bayou Goula, La. Early consideration will be given by the Corps of Engineers to the development of a dustpan dredge capable of efficient excavation to 60-foot depth for use in maintaining the recently authorized 40- by 500-foot channel in this sector of the Mississippi River.

Special Maintenance Dredging Problems

In addition to the maintenance of a dependable channel on the Mississippi River, the Corps of Engineers project provides for the maintenance of access to the off-river harbor channels at Memphis, Helena, Greenville, Vicksburg, and Baton Rouge. The siltation of the Vicksburg Harbor that followed the 1876 Centennial cutoff has been satisfactorily solved by diverting the Yazoo River past the Vicksburg city front via the lower arm of the abandoned Centennial Band. High-water siltation in the approach channels to the other four harbors requires annual dredging.

The port of Baton Rouge requires no dredging to maintain the authorized 500-foot channel through the harbor. At New Orleans, annual channel sedimentation is heavy along the convex side of the channel. Dredging is required to maintain the 35-foot- by 1,500-foot-wide channel, which is parallel to and 100 feet riverward of the wharf line.

Mississippi River Passes

The Mississippi River brings to its mouth sediment loads of 1,000,000 tons daily. As shown in figure 6, this alluvian silt advances the 35-foot-deep contour at the mouth of the

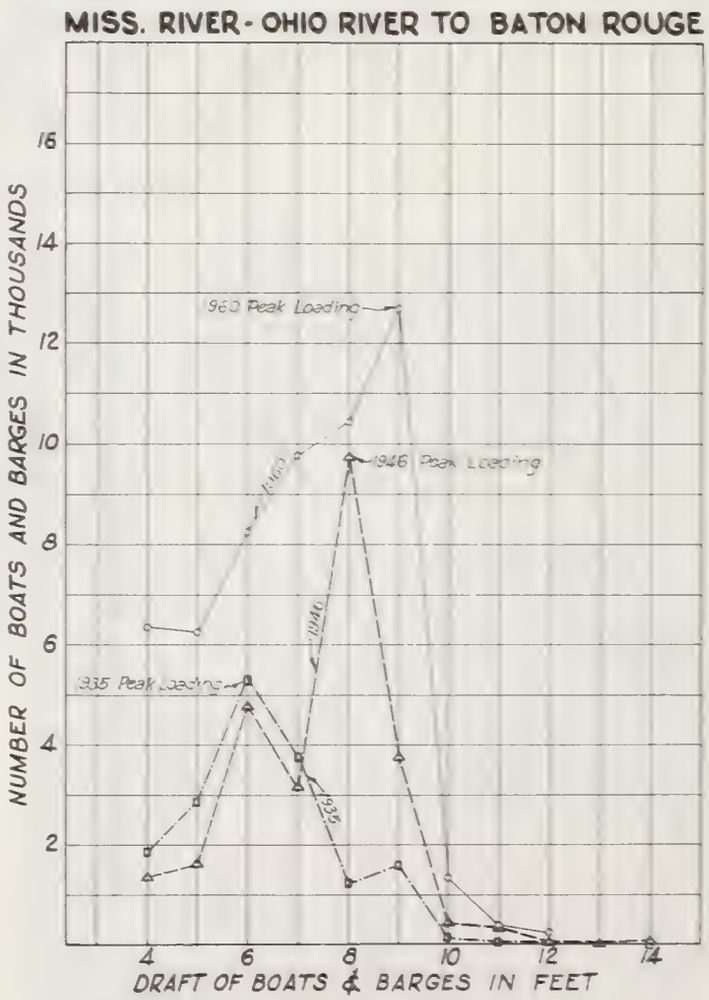


FIGURE 5. — Vessel draft increase.

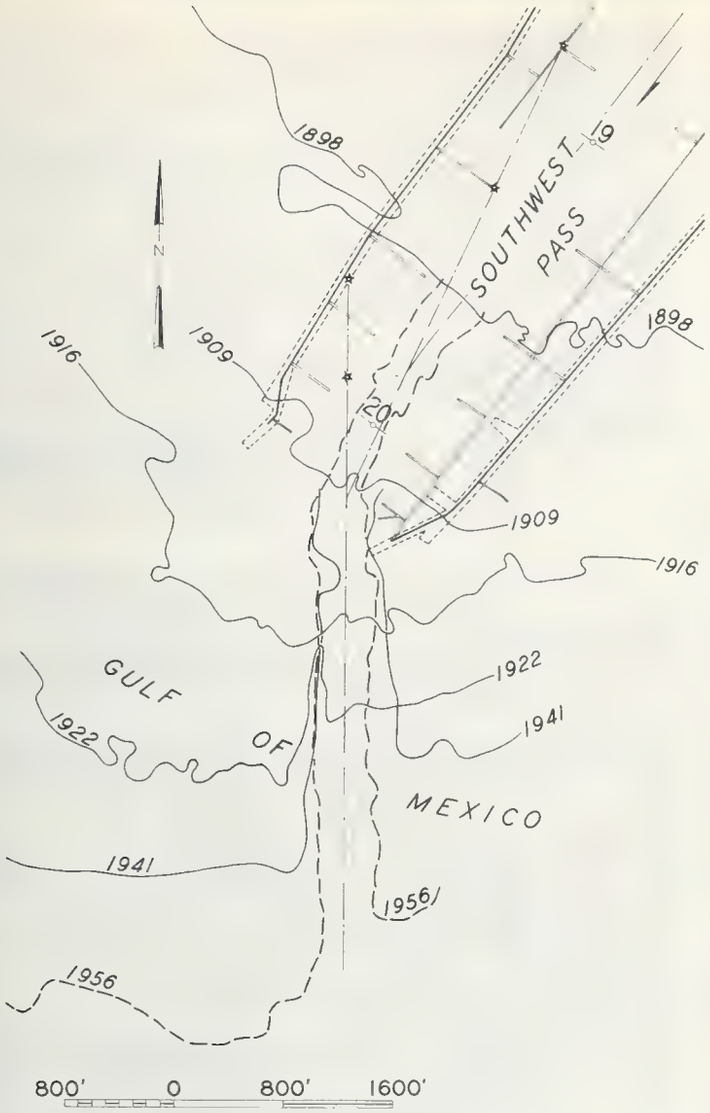


FIGURE 6. — Advance of 35-foot contour, Mississippi River delta.

Mississippi River seaward about 100 feet per year. Consequently, silt is the major factor to be considered in the planning, construction, and maintenance of a dependable navigable channel through the Passes. The planning and construction work by Eads and Corthell in 1879 at South Pass remains an outstanding example of overcoming the silt-salt water consequence. Only a nominal amount of dredging is required to maintain the 30- by 600-foot channel.

Annual siltation is important to the maintenance of the batture at the Mississippi River Passes. Where the batture becomes narrow from wave wash of passing vessels and winds, it is frequently necessary to construct outlet channels to supply siltation in the open bay in order to strengthen the batture banks on the bay side and prevent serious crevassing during high-river stages.

The Eads design of the South Pass jetties and the subsequent relocation of the entrance range across the seabar largely eliminated the silt-salt water intrusion problem at South Pass. At Southwest Pass improvements are in progress to provide a dependable 40-foot channel based on a design channel layout developed from model and prototype studies. The improvements at Southwest Pass consist of pile dikes to collect the silt and constrict the channel and realinement of the lower jettied channel and the approach channel across the seabar, to conform generally with the channel alinement at South Pass. The purpose is to provide, insofar as possible, a self-maintained 40-foot channel throughout Southwest Pass without increasing the hopper-dredge requirements resulting from high-water sedimentation at the mouth. High-water deposition in the Mississippi River Passes requires the removal of some 8,000,000 cubic yards annually.

Atchafalaya River Basin Shoaling

Deposition of sediment in Atchafalaya Basin east and west floodways has become a serious threat to the flood-carrying capacity of the floodways. The development of an enlarged central channel is in progress in order to confine the flow of the Atchafalaya River through the basin and close off side-diversion channels that are carrying silt into the floodways. Since 1954 some 115,000,000 cubic yards has been dredged or placed under contract to be dredged in developing and enlarging the central channel. About 140,000,000 cubic yards is scheduled to be dredged to complete the central enlargement and distributary closures from Alabama Bayou, mile 54.0, to Stouts Pass, a distance of 61 miles. High-water siltation in Grand Lake requires annual maintenance dredging to provide the 12- by 125-foot navigation channel.

In the sector of the Atchafalaya River between Morgan City and the Gulf, siltation in the 15-mile seabar and approach channel requires annual dredging.

Calcasieu River

The 35-mile deep-water channel (30 by 250 feet) between Lake Charles and the Gulf of Mexico was completed in 1941, and at that time maintenance was assumed by the United States. In 1953, deepening of the channel to 35 feet was completed. The project channel crosses three large lakes and courses along the margin of Calcasieu Lake for a distance of 14 miles. Siltation from stream runout, wave wash of passing ships, and extreme weather conditions has been heavy, particularly through Calcasieu Lake. Since 1953 some 29,900,000 cubic yards of material has been removed in maintaining the project channel.

The present narrow 250-foot channel is being enlarged to 40 by 100 feet to accommodate supertankers. In addition to enlargement of the existing channel, the project will provide for constructing parallel dikes through the lake sections to lessen the heavy siltation in these sectors of the waterway.

Gulf Outlet Channel

It is anticipated that siltation from wave wash of passing vessels, sea action, and the bedload movement from the littoral currents, which are westward athwart the seabar channel, will require extensive maintenance dredging to maintain the 70-mile tidewater channel now under construction from New Orleans to the Gulf of Mexico via Chandeleur Islands. The bulk of the shoaling is expected to occur in 20 miles of open-water channel through Breton Sound and across the seabar channel to the Gulf of Mexico.

Conclusions

The river engineer is concerned with two ultimate problems in the maintenance of our modern waterways: (1) the reduction in annual channel sedimentation and (2) the prevention of bank erosion by the commerce it carries. These vexing problems are aggravated by rising dredge costs and the increased volume and speed of traffic carried by the waterway.

On the Lower Mississippi River annual channel and harbor sedimentation should lessen with the completion of upstream reservoirs and stabilization of the channel below Cairo. Here the yardsticks of increasing traffic and accelerated industrial growth are excellent measures of the economic benefits being derived from the dredging required to remove the annual channel and harbor siltation.

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PHENOMENA AFFECTING IMPROVEMENT OF THE LOWER COLUMBIA ESTUARY AND ENTRANCE

[Paper No. 68]

By JOHN B. LOCKETT, *chief, Special Projects Investigation Section, U.S. Army Engineer Division, North Pacific, Portland, Oreg.*

Synopsis

This paper describes work undertaken to improve the lower Columbia estuary and entrance for navigation and discusses past concepts of phenomena controlling the regimen of this area as related to these improvements. Recent advances in technology have given birth to a new concept of tidal hydraulic phenomena that emphasizes the relation of salinity intrusion and littoral movement to the degree of shoaling experienced in estuarine areas. The findings of prototype measurements undertaken in 1959 along the lower Columbia and other endeavors to expand the knowledge of controlling phenomena, including statistical wave studies, analyses of offshore changes, studies of attrition

and accretion of adjacent shorelines, and comprehensive investigations of the distribution of Columbia River sediments are reviewed in the light of this new concept. Authorization, construction, and verification of the comprehensive lower Columbia estuary hydraulic model are discussed, as well as tests proposed for identification of controlling phenomena and for development of the most effective and economical plan of improvement. Finally, the paper summarizes the extent of present knowledge and outlines considerations for the future under conditions of controlled upland discharge resulting from contemplated headwater reservoir operation.

Introduction

The River

Physical characteristics

Columbia River rises in Columbia Lake, British Columbia, Canada, and flows in a circuitous course for 462 miles to cross the International Boundary, from where it flows generally southward 435 miles across the State of Washington to the Washington-Oregon State line, thence generally westward for 310 miles through the Cascade and Coast Ranges to empty into the Pacific Ocean near Astoria, Oreg., as shown in figure 1. The river and its tributaries drain an area of 259,000 square miles, consisting of rugged mountain ranges separated by valley troughs, trenches, and plateaus, of which 39,500 square miles lie in Canada. Elevations in the basin range up to nearly 14,000 feet above sea level in the Rocky Mountain system on the east. The total fall of the Columbia River from its source to the Pacific Ocean is 2,652 feet.

Runoff

The normal annual runoff of the Columbia River is 180,100,000 acre-feet, and the discharge pattern of the lower river is quite regular with low flows occurring from September through March and high flows from April through August. At The Dalles, Oreg., 192 miles above the entrance, the mean annual runoff is 133,700,000 acre-feet and extremes in mean daily discharge at this point have ranged from 36,000 to 1,240,000 c.f.s.

Role in economic development

Water resource development has been of basic significance in the economic evolution of the Pacific Northwest, which has in recent years emerged from an almost complete dependency on its basic resources of agriculture and timber to diversification and broadening of economic activities by production of more and more finished products. Water transportation by inland rivers, the Pacific Ocean, and the Puget Sound area has played a major role in the economic expansion of this great region. With suitable harbors that are strategically located, a flourishing trade has developed with Alaska, Canada, the Far East, California, and the east coast of the United States. The Columbia River, the most important inland waterway, provides a route for ocean-going traffic from its mouth to Vancouver, Wash., and, through the connecting Willamette River, to Portland, Oreg. Improvements are being undertaken to extend deep-water navigation with authorized channel depth of 27 feet to The Dalles. Barge traffic extends up the Columbia to Pasco, Wash., by a combination of open-river regulation and slackwater navigation provided by Bonneville, The Dalles, John Day, and McNary Dams. With completion of four authorized dams on the Lower Snake River, slackwater navigation for barge traffic will then extend to Lewiston, Idaho. Progressive improvement of the Columbia River has been accompanied by significant increases in ocean-going commerce as shown by traffic trends of major commodity groups in table 1.

TABLE 1.—*Ocean-borne commerce on Columbia River*

Item	1925	1940	1950	1955	1960
	Thousand tons	*Thousand tons*	*Thousand tons*	*Thousand tons*	*Thousand tons*
Agricultural products	733	812	695	1,153	4,417
Animal products	41	49	61	20	69
Minerals	31	86	323	71	989
Forest products	1,998	1,962	1,372	1,775	1,041
Petroleum products	1,720	3,050	5,971	7,090	5,551
Miscellaneous	713	965	792	820	426
Total	5,236	6,924	9,214	10,929	12,493

The Lower Estuary

Extent

The Columbia Estuary, as affected by tidal variations, extends during periods of low river flows up to Bonneville Dam on the Columbia River, 140 miles above the entrance, and to Willamette Falls at Oregon City on the Willamette River. At higher river stages the sections of the river thus affected by tidal conditions progressively move downstream. The reach below Oak Point, Wash. (river mile 52), shown on figure 2, forms the lower Columbia Estuary, and tidal conditions in this section generally have more effect on stages than river flows. In this reach the main river channel traverses a meandering course between generally widening bank lines, past many low islands and shoals, to eventually join the Pacific Ocean between the rugged headlands of Cape Disappointment on the north and the low-lying Point Adams on the south.

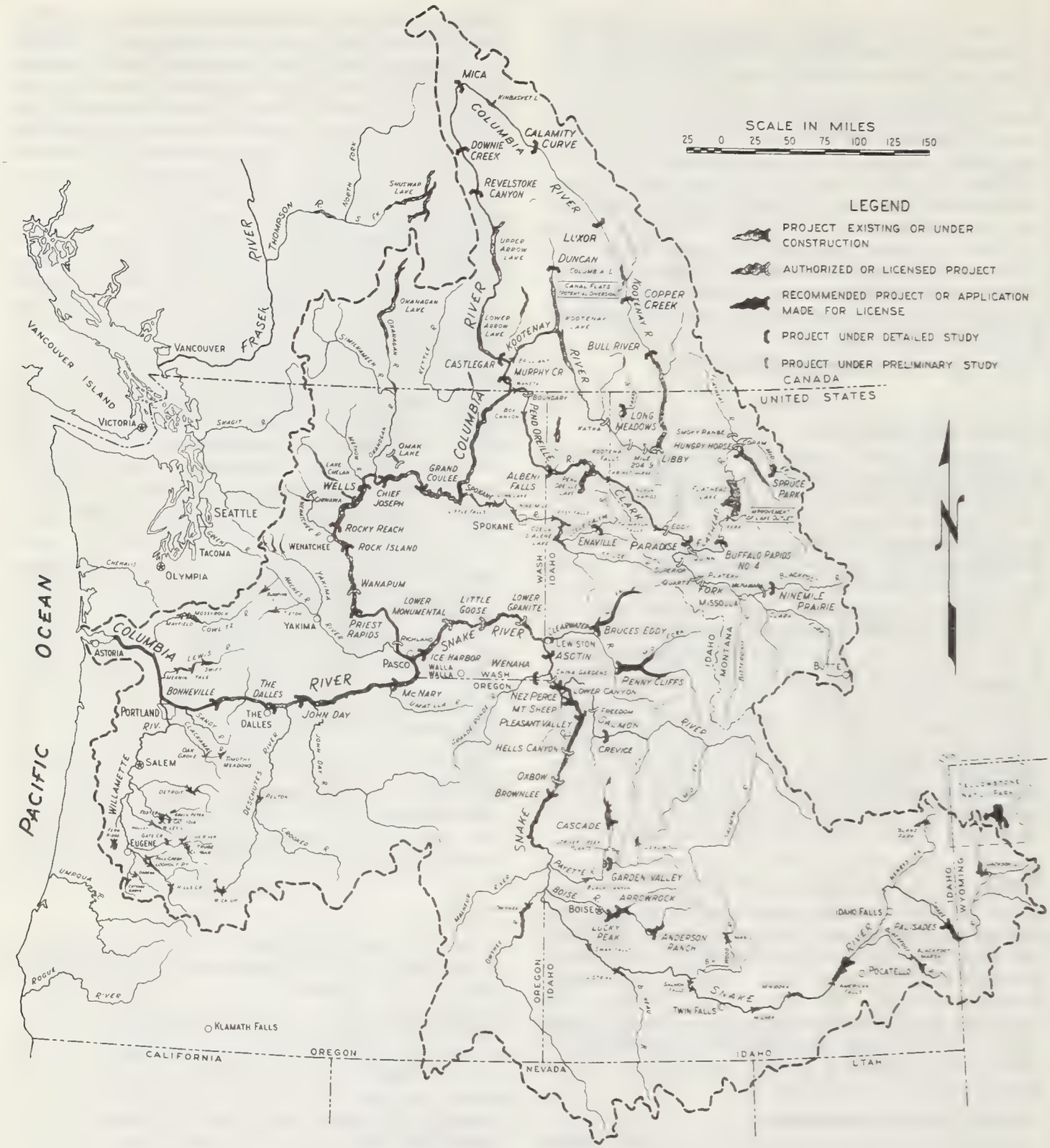


FIGURE 1. — Columbia River Basin.

Tidal influence

Normally, reversals of current direction occur throughout the lower Columbia Estuary as the result of tidal fluctuations of the Pacific Ocean. At Fort Stevens, river mile 8, the average daily tidal variation is about 8 feet. The lowest tide observed at this point was estimated to be 3.0 feet below mean lower low water and the highest stage, which resulted from a combination of storm activity and tidal action,

FIGURE 2. — Lower Columbia Estuary.

was 11.6 feet above mean lower low water. The tidal prism of the Columbia Estuary has been estimated at 732 square-mile feet.

The Entrance

General

Perhaps the most consistent characteristic of the Columbia entrance, figure 3, is its continually changing regimen that strives to create a balance between the forces of the ocean and those of the river. This balance is quite delicate, as any modification of the river forces in the entrance area, no matter how minor, is accompanied by a corresponding change in ocean forces. As a consequence, throughout the period of man's endeavor to improve conditions for navigation, the entrance has undergone many changes and has assumed many different complexions.

Tides

Tides at the Columbia River entrance have the diurnal inequality typical of the Pacific coast, with a long runout from higher high to lower low water. The mean range is 6.5 feet, the range from mean lower low water to mean higher high water is 8.5 feet, and extreme tides range from minus 2.6 feet to plus 11.6 feet, mean lower low water.

Winds and storms

Two seasons having wide variance in wind and sea conditions generally occur each year at the entrance. The season of severe storms usually extends from October to April, and storm winds, beginning generally in the south quadrant, gradually veer to the southwest and finally blow themselves out as they move around to the west or northwest. Storms are often continuous for several days and are accompanied by extremely heavy seas mostly from the southwest. During the summer season, the conditions of wind and sea are much less severe than those of the winter season. From May through September, winds from the north or northeast begin in the morning, increase in force during the day, and diminish at night. Occasionally during the winter and spring, there are several days when an easterly wind predominates and the weather is cold.

History

Prior to Improvement

A chart (fig. 4) made under the direction of Admiral Vancouver clearly indicated the existence in 1792 of a single channel at the Columbia entrance with a depth of 4½ fathoms between Cape Disappointment and Point Adams. In 1839 Sir Edward Belcher revealed the existence of two channels at the entrance separated by a large shoal area which forced the deeper channel northward against Cape Disappointment to turn sharply southward and join a generally shallower channel along Point Adams, as shown in figure 5. This shoal area, called the

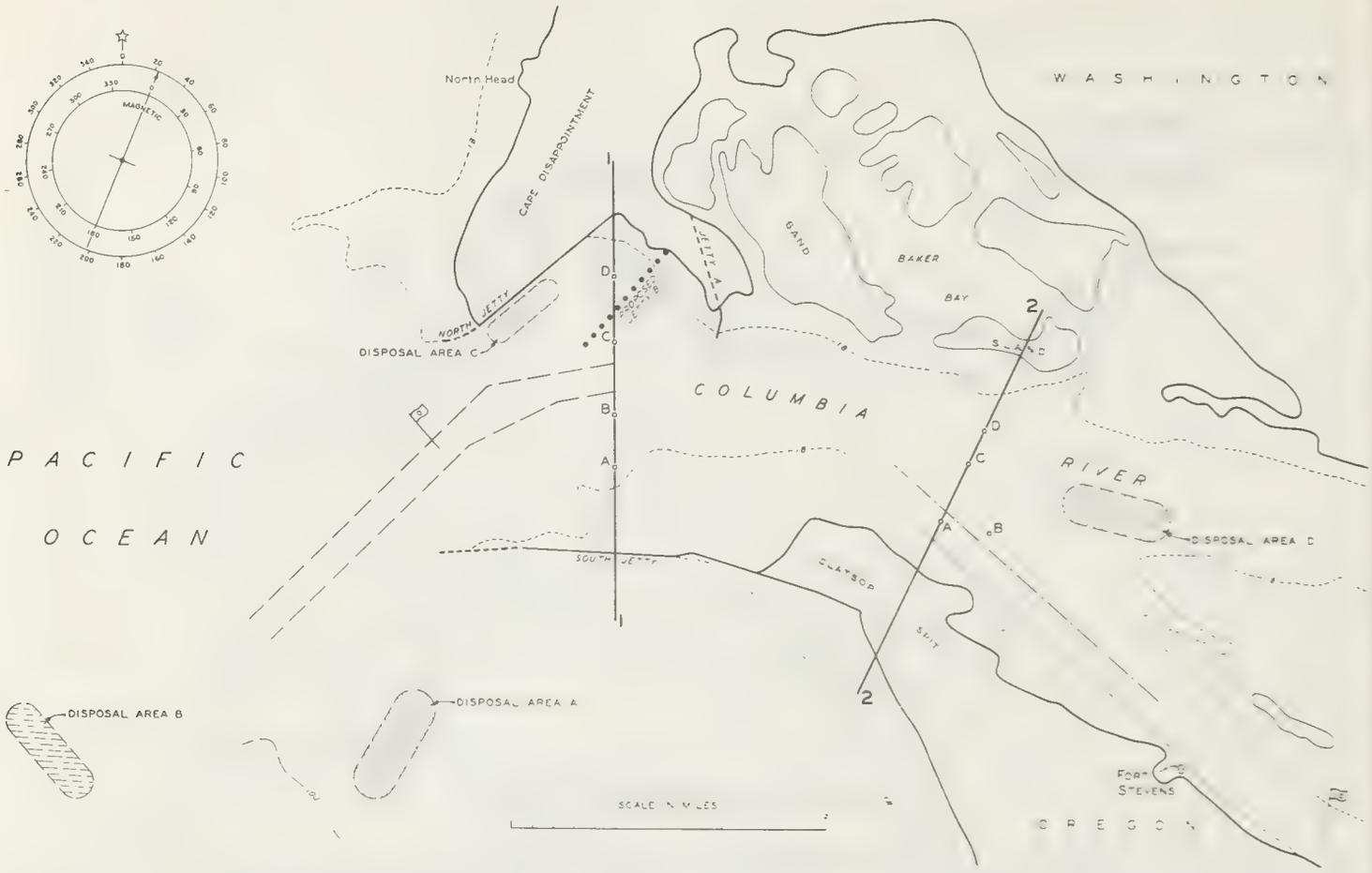


FIGURE 3. — Columbia Entrance.

Middle Sands, grew in size until its 18-foot contour in 1874 embraced a very large area seaward and south of Cape Disappointment and covered almost all of the area now occupied by the present entrance. A part of the Middle Sands formed an island inside the entrance that later separated from the main shoal area and, migrating to the north to form Sand Island, restored a single channel to the entrance in 1885. At that time the main part of the Middle Sands had moved westward to create a westward extension of Cape Disappointment and minimum depths of 20 feet prevailed on the ocean bar, as shown in figure 6.

30-Foot Entrance Project

Early attempts to improve the entrance, prior to the adoption by Congress in 1882 of the initial permanent navigation project, consisted of sporadic dredging in combination with the construction of temporary training structures. It was soon realized even in those early days that such limited work could not be effective in controlling, or even influencing to any appreciable degree, the outsized forces at work in the entrance and that improvement of more permanent character would be required. The first project was adopted on August 2, 1882, pursuant to the recommendations of the board of engineers for the permanent improvement of the mouth of the Columbia River.[1]

This project provided for the construction of a South Jetty, 4½ miles in length, lying in a generally northwesterly direction from its base at Point Adams, to secure a depth of 30 feet over the entrance bar. Construction of the rubble-mound structure was initiated in April 1885 and completed, with four groins along its northerly side, in October 1895. The crest of the completed jetty sloped from 12 feet above mean lower low water at its shore end to plus 10 feet at a point 1⅛ miles from the shore, thence to plus 4 feet at its outer end. A total of 946,000 tons of stone were placed in the structure at a total cost of $1,969,000.

Although shoals marking the beginning of Clatsop Spit began to form along the north side of the jetty during its construction, a general improvement of depths over the bar was soon evident. By 1895 depths of 31 feet were

[1] Report of the Board of Engineers, dated 13 October 1882, printed as part of Senate Exec. Doc. 13, 47th Cong., 2d Sess.

available on an alinement that, under the influence of the jetty, had migrated to the north about 3 miles. These favorable conditions were only temporary, however, as the continued

Taken from
A CHART
shewing part of the
COAST OF N.W. AMERICA,
with the tracks of His MAJESTY's Sloop
DISCOVERY and Armed Tender CHATHAM;
Commanded by GEORGE VANCOUVER Esqr. and prepared
under his immediate inspection by Lieut. Joseph Baker, in which
the Continental Shore has been traced and determined from
Lat: 45°. 30 N. and Long. 236°. 12 E. to Lat: 52°. 15 N. and Long. 232°. 40 E.
at the different periods shewn by the Tracks
(1792)
ENTRANCE ? (238°)
of
COLUMBIA RIVER
Lat: 46° 19 N.
Long. (206°) 06 E.
Var. 20. 00 E.
Gray's Bay
Scale of two Leagues
U.S. ENGINEER OFFICE.
Portland Ore. April 25, 1881.
Letter from the Chief of Engin. dated April 8, 1881.
states that that "The engraved chart bears no date,
but was published in May, 1798."
G. L. Gillespie
Maj. Engrs

FIGURE 4. — Chart by Admiral Vancouver.

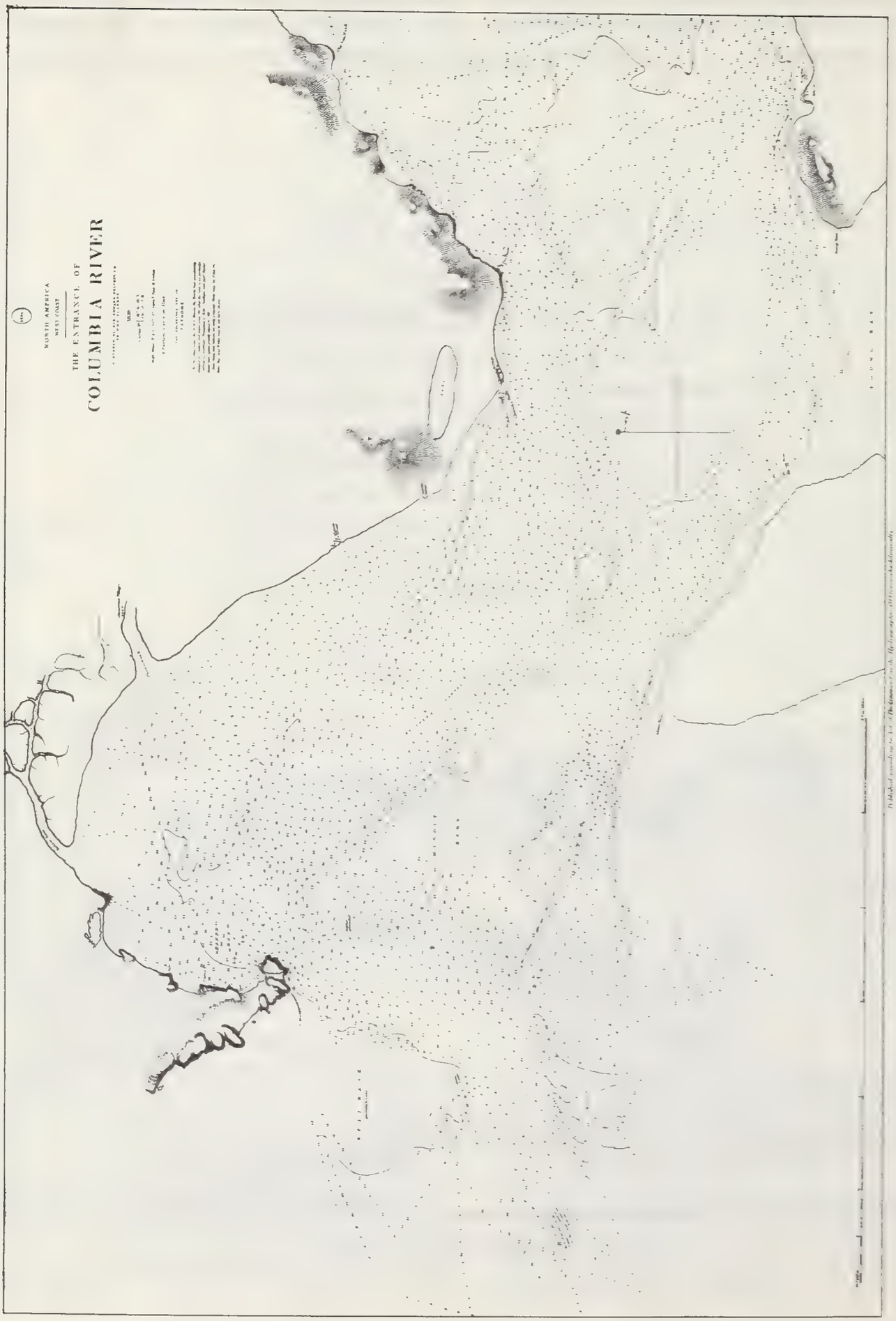


FIGURE 5.—Chart by Sir Edward Belcher.

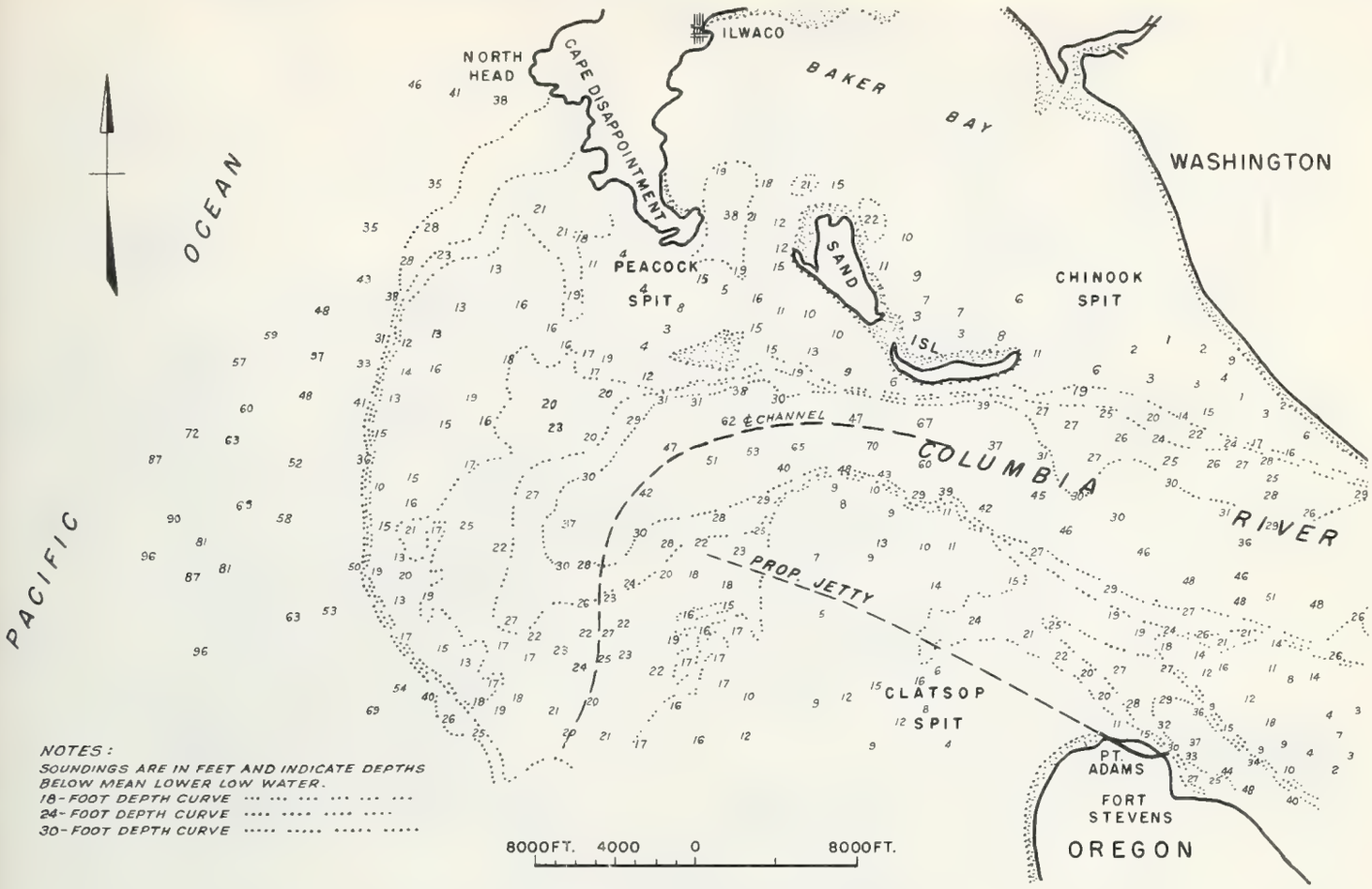


FIGURE 6. — Columbia entrance, 1885.

northward migration of the channel by 1898 reduced depths to 29 feet and by 1901 caused further deterioration of the entrance channel until in 1902, as shown in figure 7, there existed three entrance channels, each only about 22 feet deep. In the meantime Clatsop Spit had grown to form a hook-shaped island through the South Jetty at its midlength, which later extended to create a solid land connection with Point Adams.

40-Foot Entrance Project

Construction phase

In view of these deteriorating conditions and the need for increased entrance depths to furnish a margin of safety to navigation, Congress, in the River and Harbor Act of March 3, 1905, adopted the recommendations of the board of engineers on project for improvement of the mouth of Columbia River[2] that provided for extension of the existing South Jetty and construction of a North Jetty and groins, in combination with dredging, to secure an entrance channel one-half mile wide and 40 feet deep. As the South Jetty was being extended seaward, a single entrance was restored and by 1911 a depth of 24 feet was again available across the ocean bar. As completed to a total length of 6.6 miles in August 1913, the South Jetty had an average top width of 25 feet and an elevation of plus 10 feet from the shore to the "knuckle" end of the original project, from whence the height was increased by stages to an elevation of plus 24 feet at the outer end.

Construction of the North Jetty began in 1913 and by the next year, when only about 1 mile in length, entrance depths had increased to 30 feet. As constructed, the North Jetty had a top width of 25 feet, sideslopes of 1 on 1½, and crest elevation of 28 to 32 feet above mean lower low water. Completion of the jetty to its full length of 2.4 miles in August 1917 was accompanied by further improvement of entrance depths to 40 feet through a narrow bar channel. In addition to 410,000 tons of stone used for rebuilding the outer end of the original South Jetty, a total of 4,427,000 tons of stone was placed in the extension of the South Jetty, at a total cost of $5,657,000. Nearly 3 million

[2] Report of the Board of Engineers, dated 24 January 1903, printed as part of appendix 20 of Annual Report, Chief of Engineers, U.S. Army, 1903, Pt. 3, v. 3.

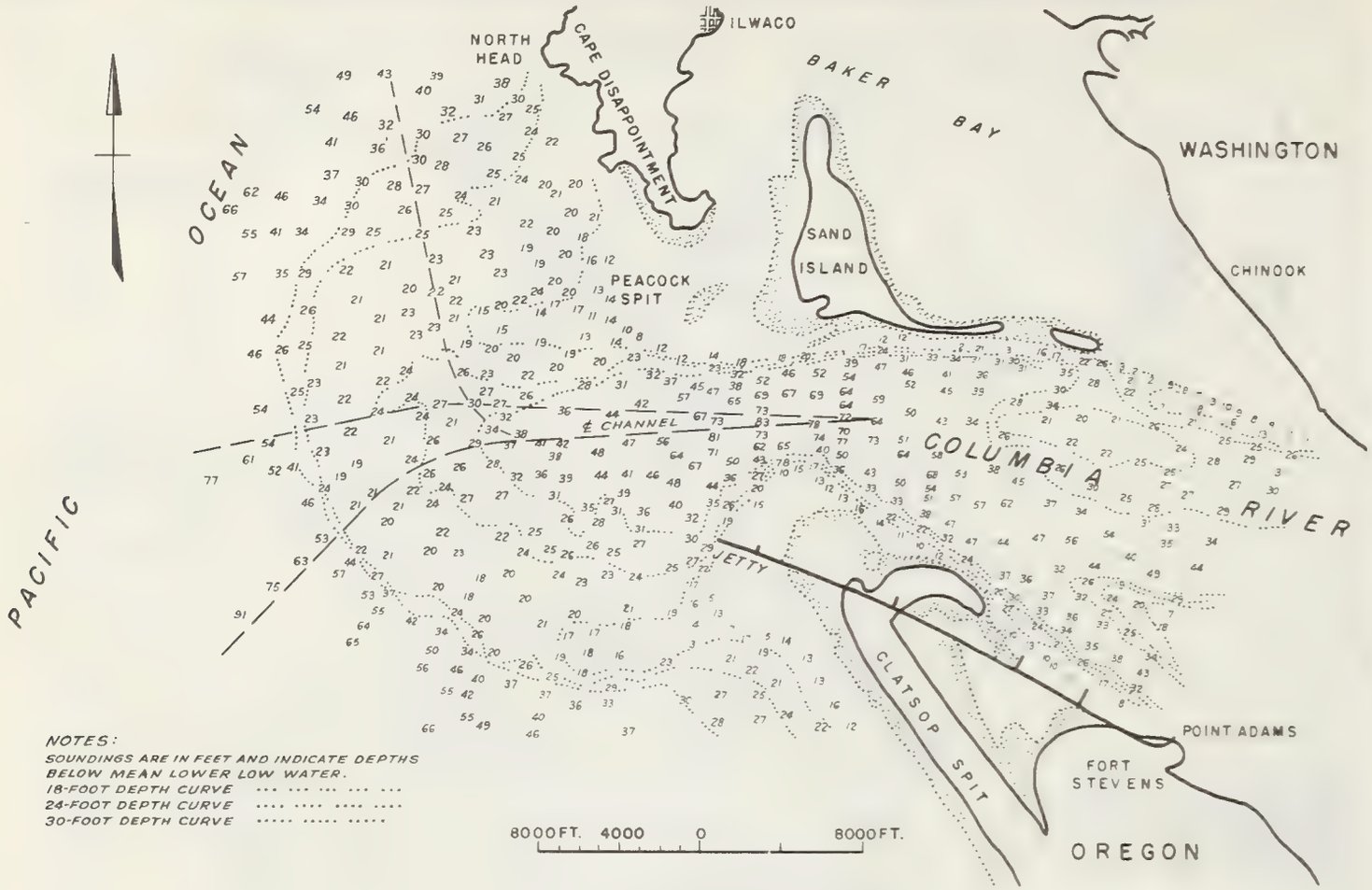


FIGURE 7. — Columbia entrance, 1902.

tons of stone were used in the construction of the North Jetty at a cost of $4,319,000.

Adjustment phase

Completion of the North Jetty, with its positive control on flows, marked the beginning of the achievement of the long-sought goal of securing dependable depths through the entrance for navigation. By 1920 the 40-foot depth contours had retreated considerably, providing an entrance channel about 1 mile wide and, during the next 5 years, further deepening occurred to make a minimum entrance depth of 45 feet available in 1925. By 1927 minimum depths of 47 feet were available over the bar, as shown in figure 8. For the first time, depths in excess of 40 feet then prevailed along the channel side of the North Jetty. However, concurrently with improvement of navigation conditions, shoals developed immediately north of the North Jetty and formed a solid land mass extending almost to its full length. Clatsop Spit continued to grow until by 1931 its westward and northward extension had reduced entrance depths to about 43 feet.

Refinement phase

Rehabilitation of the South Jetty was begun in 1931 and completed in 1936. Later in 1936, the outer 500 feet of the structure was impregnated with a hot asphaltic mix in an attempt to prevent disintegration at this point by heavy seas. This expedient did not prove successful, and a solid concrete terminal was constructed at the outer end in 1941. Further growth of Clatsop Spit was accompanied by movement of the navigation channel closer to the North Jetty and by development of increased depths along the channel side of the outer end of that structure. In an effort to prevent the continued movement of the channel to the north, four permeable dikes were constructed along the south shore of Sand Island and jetty A, extending southward from Cape Disappointment, was completed in 1939. In spite of this, the continued westward movement of Clatsop Spit was accompanied by removal of more and more of the shoal area protecting the North Jetty and some damage was caused by undercutting along the channel face of the structure.

In 1939 the North Jetty was rehabilitated and a concrete terminal block placed at its outer end. Continued advances of the Spit through 1951 coincided with definite movement of the 40-foot depth contour along the north side of the entrance channel to the east and south. In 1952, for the first time, an inner bar

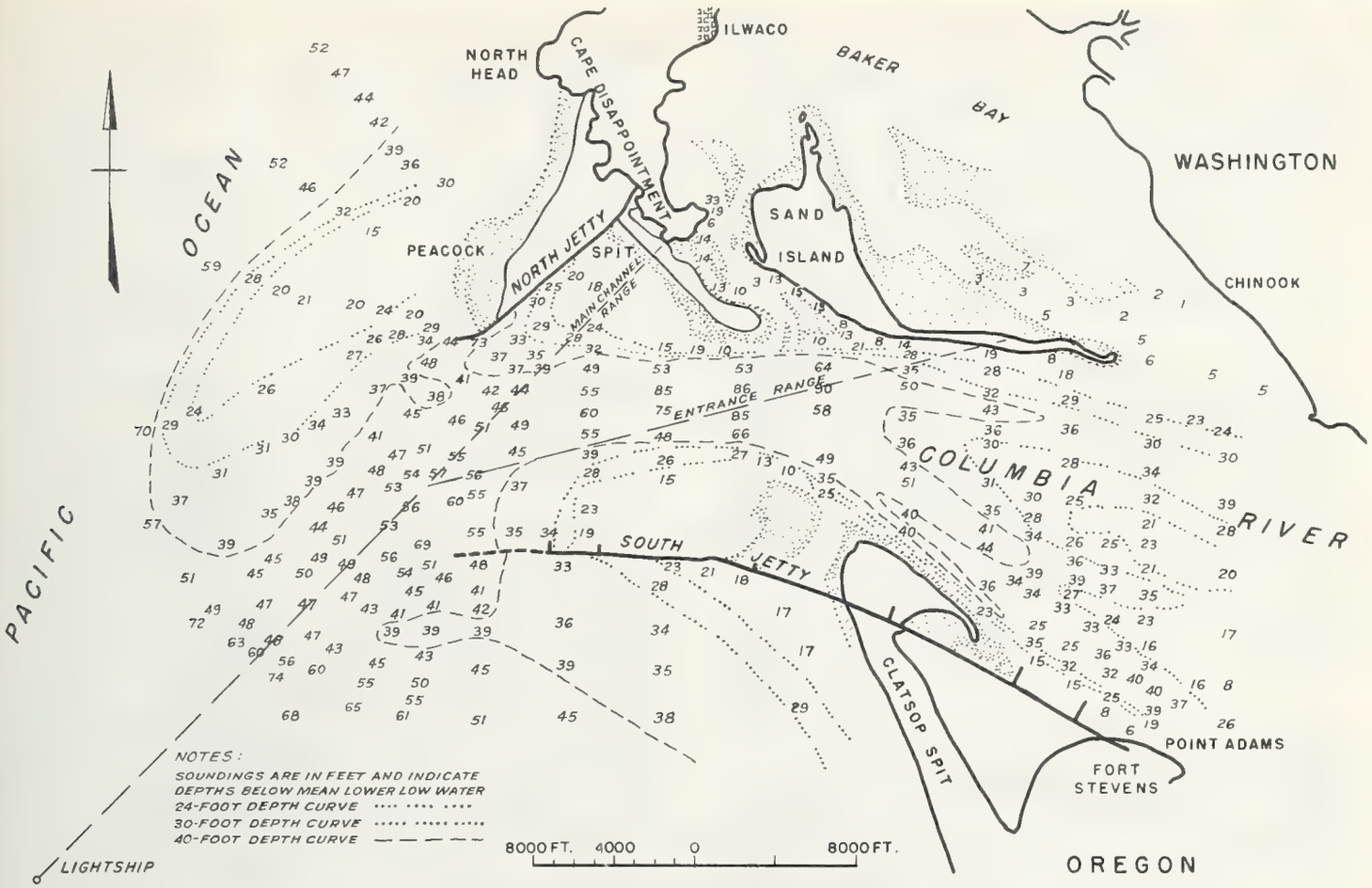


FIGURE 8. — Columbia entrance, 1927.

in extension of Clatsop Spit had formed between the jetties. Reconstruction of the South Jetty involved the placement of 4,288,000 tons of stone in the superstructure at a cost of $4,930,000. Rehabilitation of the North Jetty required the placement of 244,000 tons of stone and 234,000 tons of stone were used in the construction of jetty A, all at a total cost of $1,271,000.

Dredging

As ample depths prevailed over the ocean bar, dredging under the 40-foot entrance project was confined to the face of Clatsop Spit. Amounts dredged since 1939 under that project are shown in table 2; the full project width was obtained only during the last 3 fiscal years listed.

TABLE 2.—*Dredging on ocean bar since 1939; 40-foot project*

Fiscal year	Amount dredged	Fiscal year	Amount dredged
	Cubic yards		*Cubic yards*
1939	501,300	1949	1,023,800
1940	1,319,800	1950	923,200
1941–44	0	1951	1,000,300
1945	393,100	1952	1,267,400
1946	186,900	1953	2,796,800
1947	483,000	1954	2,141,700
1948	1,030,200	1955	1,749,300

48-Foot Entrance Project

Project progress

To meet the needs of modern ocean navigation, Congress, in 1954, modified the entrance project to provide for minimum depths of 48 feet throughout the half-mile channel width to be secured initially by dredging and, subsequently if experience so warranted, by construction of spur jetty B along the north shore (fig. 9), which also shows conditions prevailing in 1962. Dredges were successful in 1956, the initial year of dredging on the modified project, in securing by the end of the season in October depths of 48 feet over the full channel width, in spite of heavy amounts of erratic shoaling and scouring that accompanied the dredging operations.

Extremely heavy shoaling of the newly dredged channel during the following winter months, together with lack of sufficient dredging plant, resulted in the decision to concentrate the 1957 dredging effort to obtain project depths throughout a channel of only 1,500-foot width. Dredging in 1957 was also accompanied

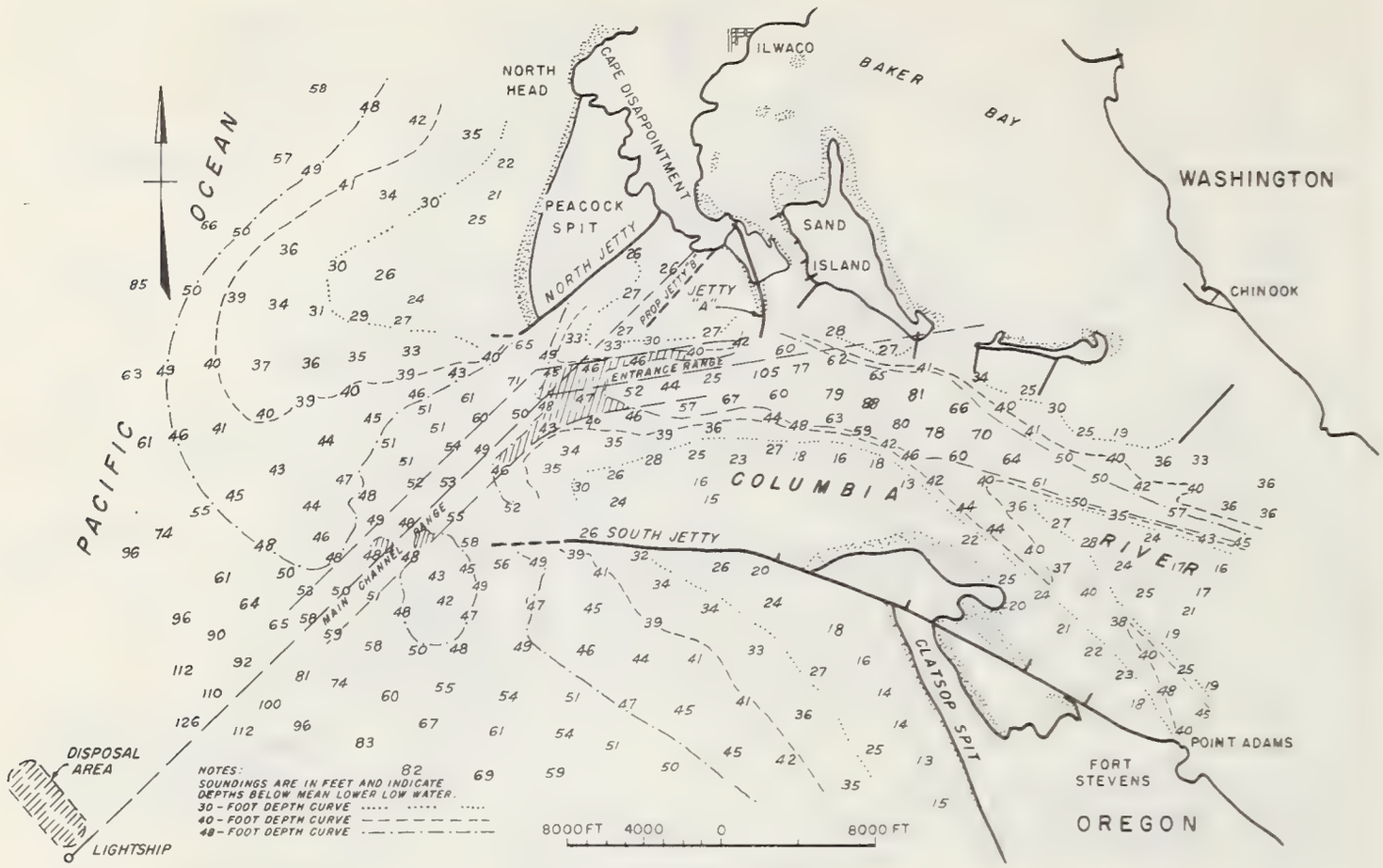


FIGURE 9. — Columbia entrance, 1962.

by erratic shoaling and scouring over the 1,500-foot channel and was similarly followed by proportionately heavy shoaling during the ensuing winter season. Approximately 75 percent of the material dredged in 1956 and 1957 was disposed in an offshore area approximately 1 mile south of the extreme end of the South Jetty (area A), as shown on figure 3, with the balance disposed in deep water along the entrance range (area B), adjacent to the inside of the North Jetty (area C), and upriver in the vicinity of Chinook Point (area D). Quantities dredged during these and subsequent years and the amounts of total apparent shoaling and scouring noted within the authorized channel width by frequent condition surveys are given in table 3.

TABLE 3.—*Dredging, shoaling and scour; 48-foot project*

Calendar year	Quantity dredged	Amount of—		Net	
		Shoaling	Scouring	Shoal	Scour
	Thousand cubic yards	*Thousand cubic yards*	*Thousand cubic yards*	*Thousand cubic yards*	*Thousand Cubic yards*
1956	14,436	8,492	6,263	2,229	.....
1957	3,909	7,279	7,436	.....	157
Total, 1956–57	18,345	15,771	13,699	2,229	157
1958	2,603	3,216	5,942	.....	2,726
1959	2,289	6,146	3,361	2,785	.....
1960	2,286	5,008	4,048	960	.....
1961	2,054	3,559	3,007	552	.....
Total, 1958–61	9,232	17,929	16,358	4,297	2,726

Modification of spoil disposal areas

Tests of pilot equipment designed to measure subsurface current directions and velocities, conducted early in 1958 in preparation for the general prototype measurement program of the following year, revealed the fact that spoil disposed in areas A and C returned to the dredged entrance channel. As a consequence, disposal practices were subsequently modified so as to concentrate disposal of dredged materials in deeper water offshore (area B) and to eliminate as much as possible the disposal of these materials south of the South Jetty and near the North Jetty. Although since that date shoaling and scouring has generally followed the previously noted pattern, the magnitude of such occurrences has been reduced somewhat from that experienced in previous years as shown above.

Rehabilitation of entrance structures

Rehabilitation of jetty A in 1961 involved the placement of about 214,000 tons of stone at a cost of about $814,000. In that same year, 213,460 tons of stone were used in rehabilitation of the upper part of the South Jetty between the land connection with Point Adams and the "knuckle," at a cost of $2,328,000. Rehabilitation of the South Jetty seaward of the "knuckle," involving the placement of 268,000 tons of stone, is scheduled for completion in 1963.

Past Concepts of Controlling Phenomena

Approach to Past Concepts

Any discussion of past concepts of phenomena controlling the regimen of the lower Columbia Estuary and Entrance, as related to the constructed improvements, should recognize the state of knowledge prevailing at that time. Engineers had little but readily observable conditions and apparent reactions of surface currents to obstacles to guide their thinking, planning, and design, of improvements, since estuarine instrumentation as it is known today did not then exist. Hence it is small wonder that some works constructed under these circumstances failed to yield the desired results. The surprising thing is that many of these works did somehow accomplish their mission, perhaps not in the exact manner predicted, and have nevertheless survived to this day as monuments to the integrity and ingenuity of their planners. Accordingly, as we review these past accomplishments in the light of present knowledge, engineers of today should feel quite grateful and, indeed humble, for the advancement in technology that alone is responsible for their more enlightened position.

Past Experience

With little to guide their efforts other than the observed conditions in the natural estuary and entrance, it was apparent to engineers charged with planning the Columbia Entrance structures that in order to provide a dependable channel, the waters of the entrance area should be confined to an area smaller than then existed in nature. To effect such an accomplishment, a South Jetty was constructed with its base at Point Adams which, while temporarily improving entrance conditions during the subsequent period of estuarine adjustment, did not fulfill the needs of then-existing navigation, as depths over the ocean bar had not been permanently improved by this expedient.

[3] See footnote 2.

[4] Mouth of Columbia River Current Survey, 1932. On file in the Office of the District Engineer, U.S. Army Engineer District, Portland, Oreg.

In view of this, engineers then studied the bankline geography of the lower estuary and concluded that further confinement of entrance flows could be effected by opposing jetties spaced about 2 miles apart at their ends, roughly the minimum distance between banklines in the lower estuary.[3] Accordingly, the South Jetty was extended and a North Jetty constructed. Although some improvement was effected by the extension of the South Jetty, this betterment of conditions was relatively minor and of a tentative nature only.

It was not until the North Jetty had been constructed that a truly dependable channel was made available in the Columbia River entrance. Although ever-changing controlling depths over the entrance bar were achieved in different phases of estuarine adjustment after the completion of the North Jetty, these depths were much greater than those that previously existed. As a result, by confinement of entrance flows to a relatively smaller area, engineers achieved a breakthrough in their efforts to control depths over the entrance bar.

Past Concept as Developed by Experience

Thus the success gained by confinement of entrance flows supported the conclusion that, in order to achieve greater entrance depths, it was only necessary to confine flows to the extent required. Although recognizing that there existed a number of observed phenomena such as formation of shoals in unexpected places, which somehow did not conform to this concept, these engineers were primarily concerned, and justifiably so, with net results as measured by past accomplishments. As a consequence, the theory of confinement of flow was accepted by these engineers as a basic principle in the planning, design, and construction of estuarine improvements.

1932 current measurements

Notwithstanding the apparent validity of the theory of flow confinement, there prevailed some question whether this theory properly integrated all factors and forces in estuarine areas. Consequently, in 1932 the Corps of Engineers undertook a program of prototype measurements at five stations across a range located some 5 miles above the ends of the jetties.[4] More than 5,600 individual current meter observations were made at several depth levels at these stations during three different stages of upland discharge. Although no instruments were specifically employed for the purpose of determining subsurface current directions, the measurements taken were interpreted to indicate that mean ebbtide velocities were greater than floodtide velocities for all stages of the upper river and, during freshet stages, were

much stronger. Also at high river stages, ebb bottom velocities were much higher than flood bottom velocities for all percentages of time. At intermediate river stages, bottom velocities were lower than at high stage, with ebb bottom velocities predominating for about half the measurements. At low river stages, flood bottom velocities were generally predominant.

If the magnitude of ebb bottom velocities as compared to flood bottom velocities and their assumed greater transporting power are considered, it appeared evident at that time that the ebb flow was sufficient to maintain a channel between the jetties of at least 40 feet deep at mean lower low water and no further contraction of the entrance appeared necessary for satisfactory channel maintenance. Thus, the results of the 1932 current measurement program were interpreted as confirming the theory of flow confinement as applied to improvements in the Columbia Estuary and Entrance.

Jetty A

Notwithstanding the indication revealed by the 1932 current measurements that further contraction works would not be necessary to secure project depths, Clatsop Spit continued to advance to the northwest, crowding the entrance channel against the North Jetty and reducing the available depths in a channel, which was at that time assuming an undesirable alinement. To correct this condition jetty A was constructed in 1938 and 1939 to confine further flows in the entrance area. Although jetty A provides a degree of protection to vessels using the west Baker Bay channel against winter winds and waves, this structure, even in combination with the concurrently constructed Sand Island dikes, has had little beneficial influence on navigation depths or conditions through the lower Columbia Estuary or Entrance.

Jetty B

Although the failure of jetty A to produce the desired channel control could not be explained at that time, the validity of the theory of flow confinement was still accepted, although to a somewhat lesser degree, by engineers as late as the early 1950's when planning for the 48-foot entrance channel was initiated. Accordingly, as part of the 48-foot project, jetty B, to be located between the North Jetty and jetty A, was recommended and authorized in 1954 as a structure to supplement and enhance the confining influence of jetty A, if initial operations to secure the 48-foot entrance channel proved impracticable. Subsequent events and almost explosive expansion of knowledge regarding estuarine phenomena have cast considerable doubt on the value of the proposed jetty B as a regulating structure, and, consequently, no further action has been undertaken by the Corps of Engineers leading toward its construction.

Modern Concepts of Controlling Phenomena

Approach to Modern Concepts

Much credit should be given to the Corps of Engineers' Committee on Tidal Hydraulics and to the Waterways Experiment Station for their pioneering efforts and studies that have led to a broadening of engineering knowledge of character and magnitude of forces controlling the regimens of estuaries and their entrances. Early hydraulic model studies at the Waterways Experiment Station included a model of the Savannah Estuary and Entrance (Georgia) and, as was the practice at that time, fresh water was employed throughout the model system. Considerable difficulty was initially experienced in attempting to verify current measurements made in the model with those observed in the prototype.

All efforts to obtain accurate verification of the model failed until someone suggested that salt be added to the ocean portion of the model in the amount necessary to simulate salinity conditions in the prototype ocean. When this was done, verification was a relatively easy task, as the model, thus operated, completely and accurately reproduced observations of current direction and velocity noted in the prototype. This "breakthrough" in model techniques led to study and definition by many engineers and scientists of the important role that salinity intrusion plays in the development of forces controlling estuarine environment. These studies and companion investigations of related phenomena by others since 1948 have produced a new concept of estuarine phenomena, which has given the waterways engineer a more complete and accurate understanding to the end that it is no longer necessary to resort to a trial and error approach to estuarine improvements.

Although this new concept has completely revolutionized all past thinking and has given birth to the embryonic science of tidal hydraulics, investigations of all estuarine phenomena are being pursued at an ever-expanding rate under the favorable technological climate prevailing in the world today. Hence, new discoveries will be made and new techniques will be developed that will further assist the engineer of the future to solve estuarine problems more completely than is now possible.

Salinity Intrusion

Although of extreme importance in the development of the estuarine regimen, the role that salinity intrusion plays in this development has to date been only broadly defined. Schultz

and Simmons [5] point out that engineers concerned with the solutions of problems in estuarine environments have become keenly aware in recent years that the amount of fresh water discharged into an estuary, and the degree to which it mixes therein with the salt water of the sea, are major factors in establishing the hydraulic and shoaling regimens of the estuary. The presence in estuaries of water of variable density causes marked differences in the magnitudes, distributions, and durations of the currents, as compared to those of a single density system.

As a result of the density difference between the heavier salt water at the seaward end of the estuary and the fresh water at the upstream end, each type of water tends to assume a rough wedge shape with the base of the wedge at the source. The interface of (or line of demarcation between) the salt and fresh water may vary from well defined to almost obscure, depending on the degree of mixing of the salt and fresh water in any given estuary. Where mixing is slight the transition from fresh to salt water is well defined and occurs within a small percentage of the channel depth. On the other hand, where the mixing is appreciable no definite interface of the salt and fresh water exists except in isolated instances.

For convenience, the degree of mixing may be classed into three broad categories of highly stratified, partly mixed, and well mixed, with the transition from one type of mixing to another being gradual instead of well defined. The most significant effect of salinity intrusion, thus defined, is the creation of density currents in estuaries that cause the bottom flood currents to predominate over the bottom ebb currents by increasing the velocity and duration of the former and decreasing the velocity and duration of the latter. The resulting net upstream movement of bottom currents within the saline region of the estuary constitutes an effective trap for sediments on and near the bottom, preventing their movement to the sea and causing the bottom to be shoaled and unstable. From their studies Schultz and Simmons arrived at the following general conclusions:

1. The degree of mixing of salt and fresh water in estuaries plays an important role in the establishment of their hydraulic regimens. As the shoaling regimens of some estuaries are related directly to their hydraulic regimens, it follows that the degree of mixing also plays an important role in establishing their shoaling regimens.

2. Because of incomplete mixing of salt and fresh water in estuaries, the predominance of flow in the bottom strata is almost always upstream while that in the surface strata is downstream; the degree of such predominance is dependent on the degree of mixing, being most prominent in the highly stratified estuary and least prominent in the well-mixed type.

3. Changes in upland discharge, tidal prism, and physical configurations of estuaries will frequently change the degree of mixing of salt and fresh water therein and thus affect such important features as the vertical distribution of current velocities, the direction and degree of flow predominance, the amount of shoaling, and the location of major shoal areas.

4. As lightweight sediments are supplied to estuaries principally through the medium of upland discharge, it follows that such discharge plays a dual role in estuarine sedimentation.

5. The magnitude of changes in upland discharge into estuaries usually far exceeds that of changes in tidal prism of physical configuration. For this reason and because upland discharge is of primary importance, it follows that major changes in upland discharge should be accomplished only after consideration of all probable effects.

Continuing Research on Salinity Intrusion

An analytical investigation of salinity intrusion and related phenomena was initiated by the Corps of Engineers Committee on Tidal Hydraulics in January 1954. This general investigation was designed to determine for conditions of open channels subject to salt water intrusion and tidal oscillations from the sea the following four aspects of the phenomena: (1) The extent of salinity intrusion and the mean salinity distribution, (2) the vertical mixing of fresh and salt water and the resulting vertical salinity distribution, (3) the vertical distribution of current velocities as affected by salinity distribution, and (4) the movement and deposition of sediments as affected by density-current phenomena. On the basis of studies undertaken on phase (1) of this general investigation, Ippen and Harleman [6] indicate that it is possible to make quantitative predictions regarding salinity intrusions in partially or well-mixed estuaries of essentially uniform cross section without prior knowledge of any existing salinity conditions. Further, the factors that cause changes in salinity intrusions have, for the first time, been quantitatively evaluated. Thus by means of intrusion equations, the effect of changes in the fresh water discharge, channel depth, etc., can be predicted. Of even greater importance is

[5] SIMMONS, H. B., and SCHULTZ, E. A. FRESH WATER-SALT WATER DENSITY CURRENTS, A MAJOR CAUSE OF SILTATION IN ESTUARIES. Corps of Engineers Committee on Tidal Hydraulics, Tech. Bul. 2. April 1957.

[6] IPPEN, A. T., and HARLEMAN, D. R. F. ONE-DIMENSIONAL ANALYSIS OF SALINITY INTRUSION IN ESTUARIES. Corps of Engineers Committee on Tidal Hydraulics, Tech. Bul. 5. June 1961.

the possibility that the diffusion and tidal parameters developed in this study will have important significance in understanding the intrusion mechanics of estuaries of nonuniform geometry. Analyses will be made of phases (2), (3), and (4) of this investigation in the light of findings of studies of phase (1) as may be applicable.

Shoaling Processes

Much remains to be learned regarding the processes involved in the formation and stabilization of shoals in estuarine channels. In 1957 the Committee on Tidal Hydraulics launched a broad program of studies considered necessary to develop essential knowledge of these processes. The different phases of work contemplated under this program are summarized as follows.

Basic laws for movement and deposition of muddy sediments

Since most estuarine shoals in the United States are composed of muds rather than sands, a contract has been made with the University of California to study the rheological properties of consolidating sediments. Natural sediments from a number of estuaries are being furnished to the university as a basis for the studies.

Effects of a repetitive scour and deposition on sedimentation

As it appears that successive scour and deposition of sediments may play an important role in shoaling processes, early flume tests to determine the significance of these factors are planned.

Techniques for radioactive tracing of sediment action

These techniques offer a great potential in the development of knowledge in shoaling processes, making it mandatory that their use be encouraged in connection with specific investigations and that consultation and advice be extended to those engaged in this work. Field tests planned by the district engineers of Galveston and Norfolk will be closely followed.

Development of in-place turbidity meter

Because of the great areas embraced by most estuaries and the constantly changing conditions of tide and fresh-water inflow, it is apparent that complete and simultaneous coverage of such systems for measurements of suspended sediment concentrations, with only approximately accurate results, will yield much more valuable information than will long-term coverage and highly accurate results. Available evidence indicates that an instrument operating on the basis of light extinction as a measure of turbidity may fulfill these needs. Development of such an instrument is under way at Johns Hopkins University.

Flocculation

Flocculation of suspended and dissolved solids plays an important role in shoaling processes, as sea water is an efficient flocculating agent, but little work of scientific value has yet been accomplished to evaluate its significance. A comprehensive literature survey of this subject has been made and a report thereon completed. Recommendations for further work in this field are now under review, and it is probable that further research will shortly be initiated.

Stabilization of deposits

The physical and chemical changes that occur in sediments after deposition appear important to an overall appraisal of the matter of shoaling processes. A literature survey of this subject has been completed and additional research is forthcoming.

Analysis and correlation of prototype data

Proper analysis and correlation of existing prototype hydraulic, hydrographic, and other data will reveal certain relations among estuaries that should lead to a better understanding of the overall subject of shoaling processes. Data on important estuaries are now being assembled and analyzed, and it is anticipated that direct comparisons and correlations should reveal definite characteristics.

Shoaling in slips and tributary channels

It is anticipated that test facilities for general investigations of hydraulic and shoaling phenomena in slips and tributary channels will shortly be constructed as additions to existing hydraulic models at the Waterways Experiment Station and tests initiated to determine the factors involved.

Classification of sediments

Proper classification of the sediments that contribute to shoaling of the different estuaries in the United States is of importance from the viewpoint of a comprehensive understanding of shoaling processes. This effort is being carried out concurrently with the analysis and correlation of prototype data previously described.

Littoral Processes

Waves of all character and magnitude breaking on a coastline generate movements of beach materials in the alongshore component direction of the generating waves. These movements establish the littoral regime of the shoreline, and the intensity of these movements determine the quantity of material or littoral drift moving past a point on the shoreline. As, during the course of an extended period of time, waves attack a shore generally from one predominant direction, so too will the littoral movements of material passing a point on that shore move in

one predominant direction parallel to the coast. Just as waves also attack this shore from other directions over an extended period of time, though less predominately from those directions, so too will there be littoral movements past the point that may be in a direction opposite that of the predominant littoral drift.

Thus, from a point along a shoreline, such as the entrance to an estuary, the total quantity of littoral material in movement is the sum of all littoral movements in both the up-coast and down-coast directions. Both of these material movements contribute to shoaling in major estuarine entrances. They also affect the delicate balance between erosion and accretion of the shoreline. In instances where littoral drift is intercepted by coastal structures such as groins or jetties, the shore updrift of these structures will grow seaward to a point, fed by the cessation of the littoral movement, while the shore downdrift from the structures will recede, being starved by the intercepted littoral drift.

Prototype Measurement Program of 1959

Background

In view of difficulties experienced in maintenance of the 40-foot entrance project resulting from the continual encroachment of Clatsop Spit shoals on the adopted channel alinement, the problem at the Columbia River Entrance was re-approached in 1956 in the light of new concepts and expanding knowledge of tidal hydraulic phenomena concurrently with initiation of work on the newly authorized 48-foot entrance project. In making this new approach, the advice and guidance of the Committee on Tidal Hydraulics formed the basis for an intensive investigation of several aspects of the problem.

As the initial step in this investigation, the committee recommended that a program of prototype measurements of current velocity and direction, as well as salinity, be undertaken to define broadly the density current regime established by environmental river, ocean, and hydrographic conditions prevailing in the lower estuary. With effects of density currents thus defined it was the view of the committee that any additional structural measures designed to reduce shoaling in the entrance area, such as jetty B, could be properly analyzed and evaluated in a hydraulic model study.

General Scope of Program

In order to achieve the above goal, it was necessary that the program provide data sufficient to meet the design and verification needs of a hydraulic model study. To this end the program required 3 cycles of measurements to obtain data for conditions of low, normal, and high river discharge. Each cycle was to include observations of current direction and velocity for a continuous full tidal period of about 25 hours at 23 stations located along 7 ranges across the lower 52 miles of the estuary (fig. 10). At each station current velocity and direction measurements were taken at 30-minute intervals at the surface and near the bottom and at the intervening quarter-points of depth.

Initially, simultaneous observations were made at one station on each range for a continuous period of about 25 hours to establish the relationship between the ranges. This was followed by simultaneous observations at all stations on each range until all observations had been obtained. Salinities and temperatures were also observed throughout the vertical at each station concurrently with the measurements of currents and directions. This program, due to its magnitude and the complexities involved in instrumentation, administration, and operation, not to mention the trying conditions of weather and heavy seas, represented a monumental task. A total of approximately 26,000 observations were taken during the 3 cycles of measurements. However, in spite of the magnitude of work involved and the difficulties experienced, reliable measurements were obtained.

Program Instrumentation

Velocity–azimuth–depth assembly

Equipment being tested by the U.S. Geological Survey to measure tidal flow was used as a guide in the prototype measurement program for the development of an instrument designed to measure subsurface current velocity and direction, as well as depth of the instrument above the bottom. This instrument consisted of a transducer for a Raytheon Fathometer, mounted in the bottom of 140-pound streamline brass weight with fins, suspended on the bottom of a Price current meter hanger bar. Also mounted in the weight was a Magnesyn compass transmitter that monitored on a remote panel the direction in which the assembly was facing as deflected by the current. A seven-conductor, pancake swivel was mounted between the current meter hanger bar and the seven-conductor, ¼ inch diameter, suspension cable. The assembly as finally developed and used in the program is shown in figure 11.

Salinity–temperature assembly

Although consideration was given to the advisability of taking water samples at the five elevations of each station and determining salinity by laboratory tritration procedures, it was felt that, in the interest of reducing the workload, a less exact method of measuring salinity would suffice for the needs of the data collection and analysis program. It was also considered

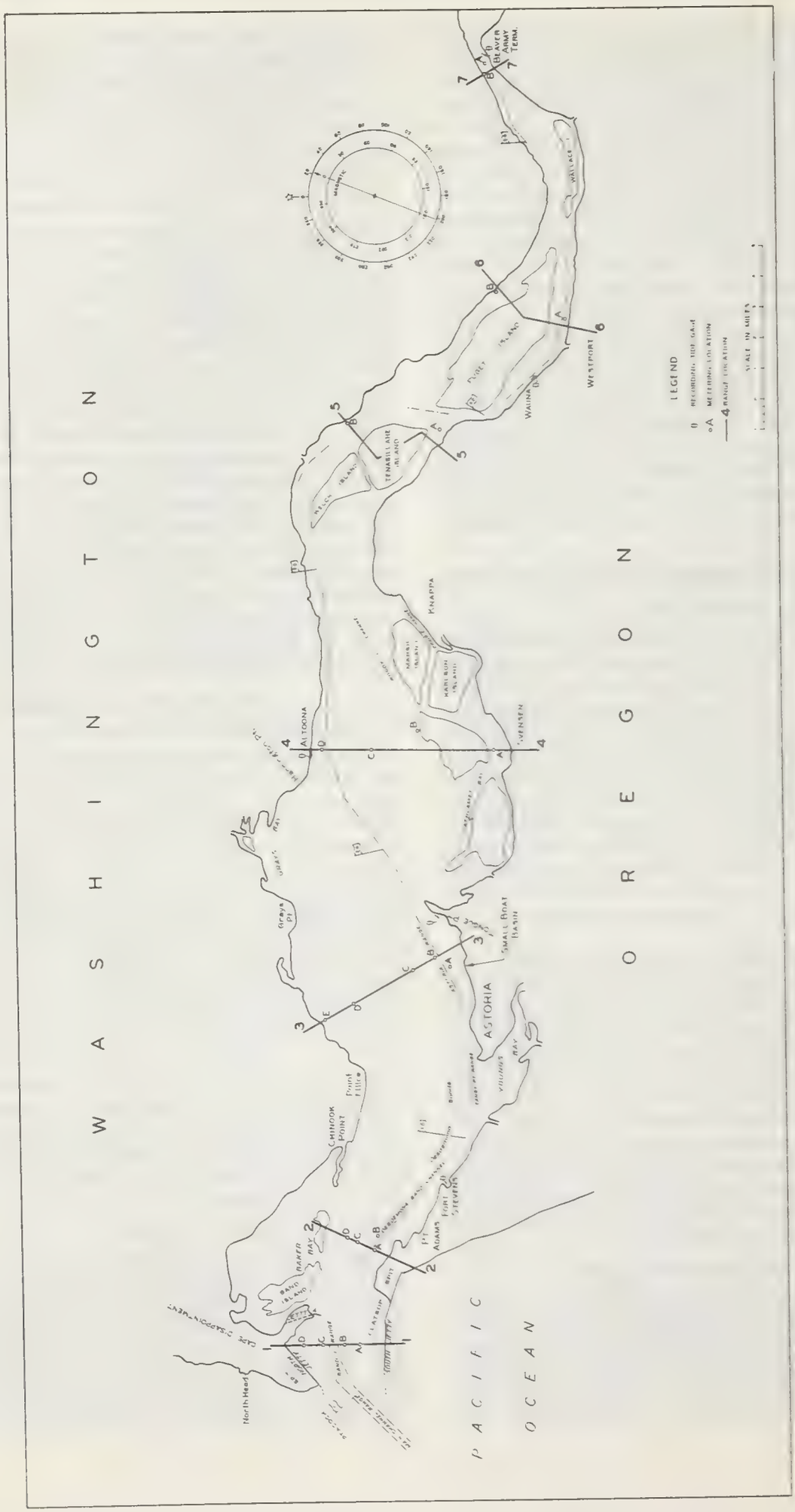


FIGURE 10. — Prototype measurement ranges and stations.

FIGURE 11. — Velocity — Azimuth — depth assembly.

advisable to obtain concurrent measurements of water temperature. To meet these requirements a conductivity cell with remote-reading resistance indicator was used to measure salinity, and a temperature cell, together with Wheatstone-balanced bridge, was used to obtain temperatures. The conductivity and temperature cells were mounted on a bracket above a 140-pound brass weight, and short leads from the cells were connected to leads from the suspension cable inside a short stainless steel hanger that was clamped to the lower end of the suspension cable, as shown in figure 12. The soldered connections were embedded in a hard, waterproof plastic filler. A seven-conductor suspension cable identical to that used for the velocity-azimuth-depth assembly was used.

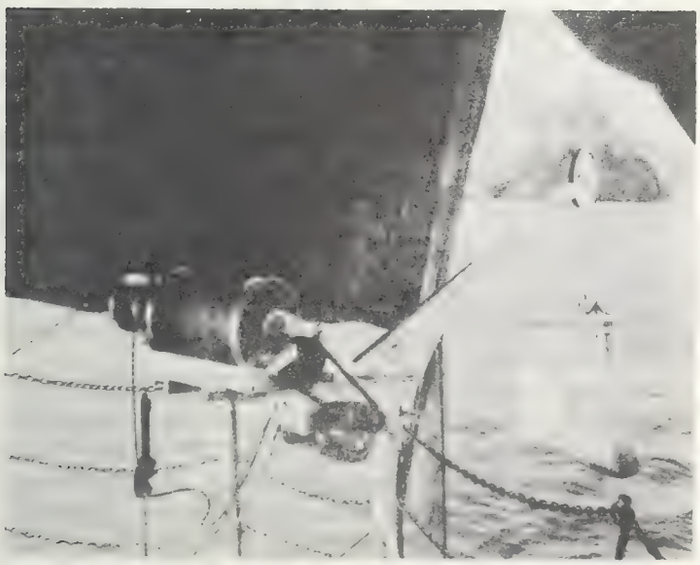


FIGURE 12. — Salinity — temperature assembly.

Pillar crane

As the metering and salinity-temperature assemblies were to be used over the sides of small boats, a special crane was designed and fabricated to raise the bottom of the assembly weights 30 inches above the boat deck and far enough away from the side of the vessel so that the danger of striking the assembly instruments against the boat hull would be minimal. Pipe was used for the main structural members of the pillar crane shown in figure 13.

Other equipment

In addition to 12-volt batteries for each metering vessel, one complete set of current metering and salinity-temperature assemblies and a pillar crane were kept in readiness for use. Automatic tide gage recorders were maintained at strategic points in the lower estuary to record tidal stage variations during the data measurement periods.

Program Administration

General

Administrative tasks associated with the program included the procurement of necessary equipment, rental of vessels, training of equipment operators, and coordination of program details with fishery and navigation interests.

Procurement of equipment

Assemblies and pillar cranes previously described were obtained by contract with local manufacturers. Small items, including repair parts and flashing lights for buoys, as well as battery chargers, were obtained by direct purchase. Storage batteries were rented for each of the measurement periods. All equipment was assembled at the Government moorings in Port-

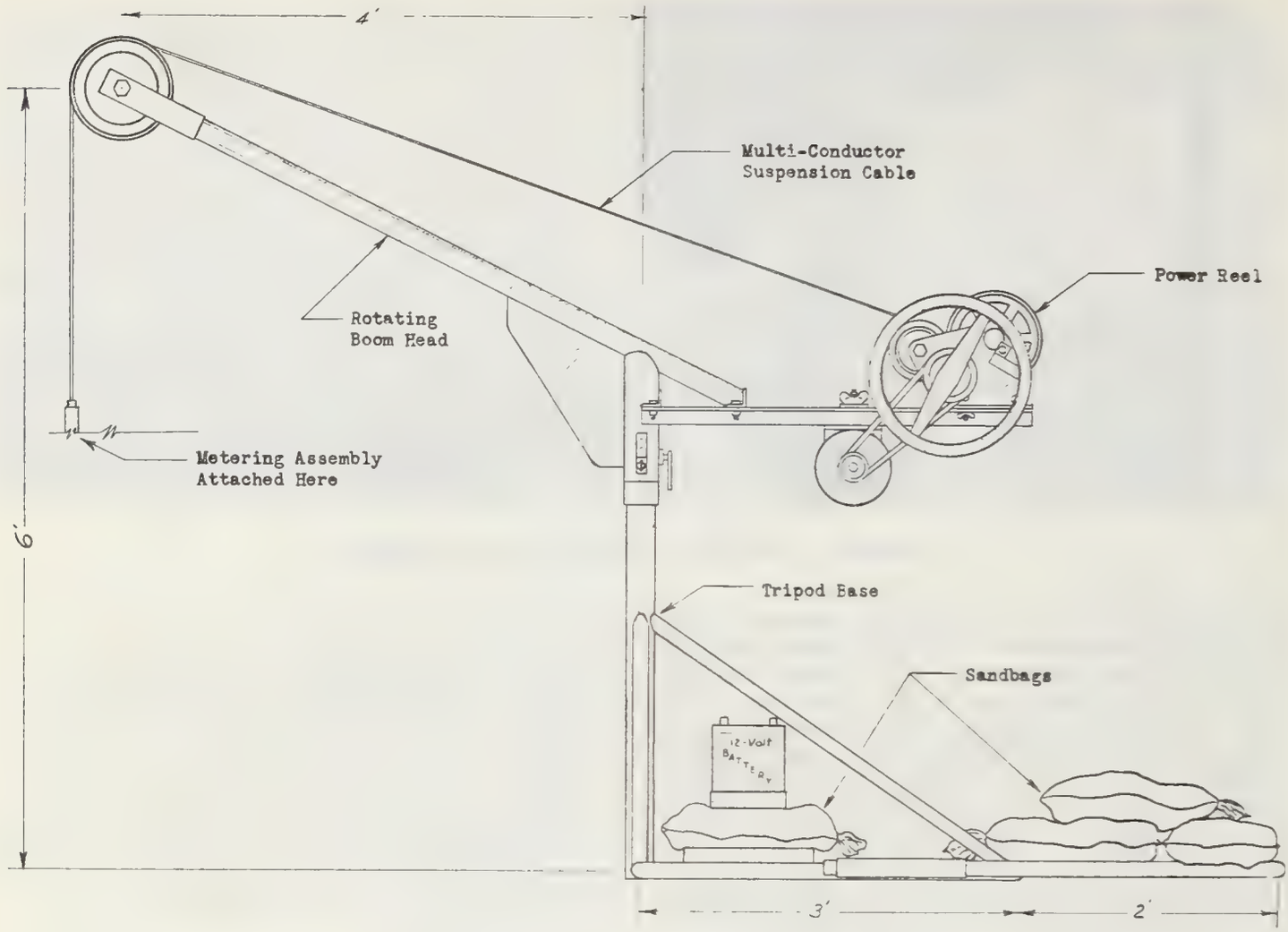


FIGURE 13. — Pillar crane.

land prior to movement by truck to the small-boat basin at Astoria, which served as the field headquarters for the program. Specifications were prepared to meet the particular requirements of the control boat, the master metering boat, and four metering boats. These vessels were hired through normal contractual procedures on an hourly rental basis.

Operator training

Professional engineers and engineer technicians from the engineering division of Corps of Engineers, Portland District, were selected as equipment operators and indoctrinated by means of a special training program. Each operator was furnished a 20-page brochure describing the purpose of the prototype measurement program and related administrative and technical details, prior to a general orientation session to insure a thorough understanding by all of their specific duties. This was followed by a 4-hour session in which each operator was taught how to assemble, operate, and maintain each item of measuring equipment.

Coordination

Discussions were held with the United States Coast Guard to determine the type of buoy most desirable to mark the location of each measurement station in the lower estuary, and to insure the availability of such buoys and needed appurtenant equipment. At the request of the Columbia River Fisherman's Protective Association, arrangements were made to forego the use of buoys at stations C and D on Range 2, to eliminate interference with fishermen's nets on established drifts and to remove and replace buoys at stations D and E on Range 3 during and after the August fish runs.

Some minor shifting of station locations in the back channels was made at the suggestions of tug and barge operators to avoid accidental removal of buoys by log tows. The Columbia River Bar Pilots Association was notified of proposed activities, and the Coast Guard was furnished the latitude and longitude of each buoy for publishing in its notice to mariners well in advance of the placement of buoys.

Program Operation

Field operations, with the attendant requirement for extreme accuracy and timing of observations, occasionally in the face of adverse weather and wave conditions, represented the most difficult phase of the measurement program. Initial operations involved the placement of first class can buoys by the Coast Guard early in April 1959 to mark the location of the measurement stations.

Distribution of vessels

Four equipment operators were assigned to each of the six vessels providing two 2-man operator crews for alternate 12-hour shifts throughout each 8-day measurement cycle. The control boat, the *Sujan,* in addition to representing an immediate source of supply of spare equipment and the means for supervision and control of operations, was also used to ferry operator crews to and from shore bases established at Fort Stevens, Astoria, Svenson, Knappa, and Westport for the purpose of reducing the time required for changing shifts. Station A, Range 3, the master-metering station, was occupied by the metering boat, *Catana,* continuously throughout each 8-day measurement cycle. The four other metering boats, the *Morning Star, Rosie, Mary K,* and *My Boat,* were phased among the remaining 22 stations to obtain combinations of 25-hour measurements at each station as shown in table 4.

TABLE 4.—*Distribution of metering boats*

25-hour metering period	Boat and station occupied				
	CATANA	MORNING STAR	ROSIE	MY BOAT	MARY K
1	3–A	3–C	3–B	3–D	3–E
2	3–A	1–A	1–B	1–C	1–D
3	3–A	2–A	2–B	2–C	2–D
4	3–A	4–A	4–B	4–C	4–D
5	3–A	5–B	5–A	6–A	6–B
6	3–A	5–B	6–A	7–A	7–B
7	3–A	4–D	2–C	5–B	7–B

Observations

Measurements of current velocity and direction, salinity, and temperature at five different levels were made every half hour at each station. These levels were located 3 feet below the water surface, 2 feet above the bottom, and at the intervening ¼-, ½-, and ¾-depth levels.

Cycles of measurement

The initial cycle of measurement was undertaken from May 5 through May 13, 1959, a period of normal river flow, when the discharge ranged from 365,000 to 404,000 c.f.s. in the lower estuary. The second cycle of measurement was accomplished between June 16 and June 24, 1959, a period of high river flow, when the discharge ranged between 532,000 and 577,000 c.f.s. The final cycle of measurement was made between September 15 and September 23, 1959, a period of low river flow, when the discharge ranged between 153,000 and 214,000 c.f.s.

Analysis of Prototype Data

General

An office analysis of observed prototype data was undertaken by the district engineer, Portland, and a record of all observations obtained and his analyses of these data are contained in a four-volume manuscript report [7] that provides the most complete source of observed data relating to the current, salinity, and temperature regimes of the lower estuary and entrance. Due to weather and sea conditions, which precluded the undertaking of a program of measurements near the point where Clatsop Shoals encroach upon the entrance channel, all observations were made upstream from this problem area.

While the measurement program did not reveal conditions prevailing in that area, it did show the vagaries of currents and other measured phenomena throughout other parts of the lower estuary and, thus, provided data for competent verification of a hydraulic model of the entrance and lower estuary. In such a model, physical phenomena contributing to shoaling in the problem area are now in the process of being observed, understood, and analyzed to the extent feasible by tests of possible structural expedients.

Flow predominance

In analyzing observations of current direction and velocity, plots similar to that shown in figure 14 were prepared for each station. Changes in current direction between flood and ebb and velocity were noted at each depth level throughout each observation cycle of approximately 25 hours' duration. From these plots it was possible, by computing the subtended areas above and below the zero velocity line, representing the volumes of flood and ebb flows, to determine the predominant direction of flow and the degree of predominance of such flows at those levels.

Figure 15 shows changes in flow predominance noted at all stations and levels during each river discharge cycle of measurement on Range 1, and figure 16 shows similar information obtained from observations taken on Range 2. From similar data developed along each measurement range it was possible to obtain flow predominance profiles along the northerly

[7] DISTRICT ENGINEER, U.S. ARMY ENGINEER DISTRICT, PORTLAND. INTERIM REPORT ON 1959 CURRENT MEASUREMENT PROGRAM, COLUMBIA RIVER AT MOUTH, OREGON AND WASHINGTON. 1960.

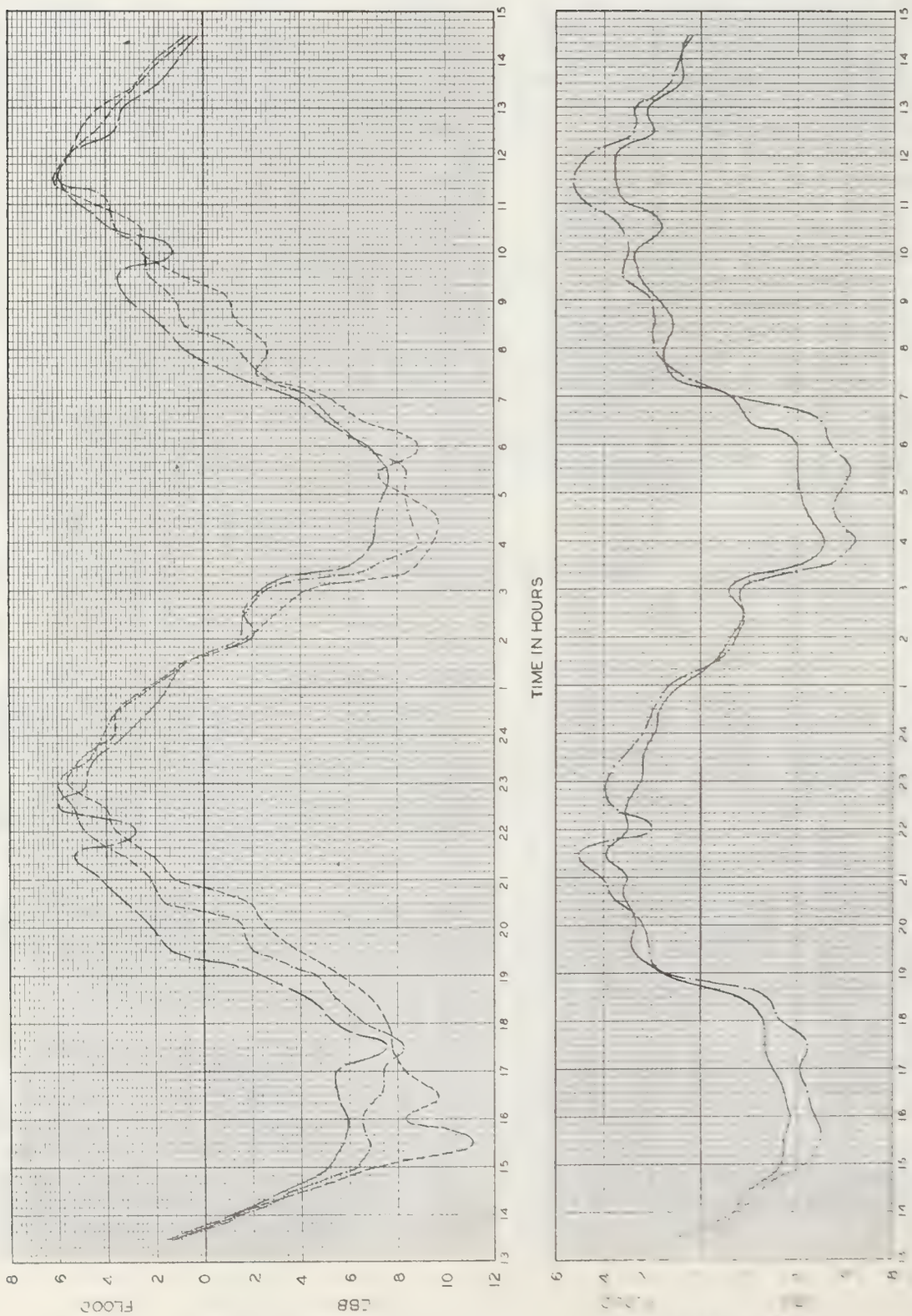


FIGURE 14. — Typical velocity — time — direction curve.

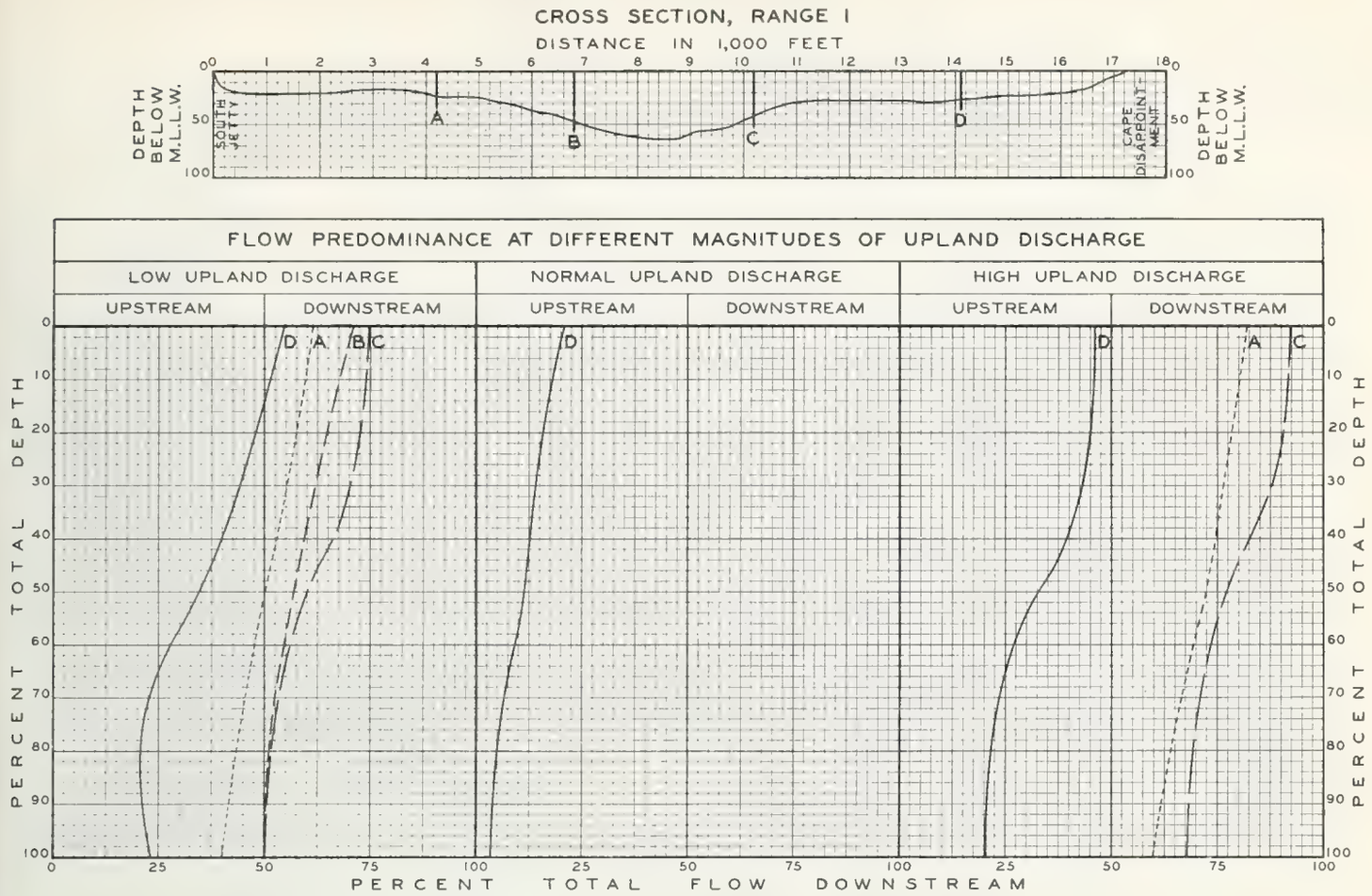


FIGURE 15. — Flow predominance, range 1.

part of the lower estuary, as shown in figure 17, as well as along the adopted ship channel alinement shown in figure 18.

Salinity intrusion

Measurements of salinity, taken concurrently with those of velocity and direction, revealed significant intrusion by salinity within the lower 20 miles of the estuary, shown in figure 19.

Temperature

Although a general trend of correlation between salinity and temperature was noted, this correlation was not conclusive, owing possibly to the time lag between the taking of the salinity and the temperature observations.

Findings of the Program

The prototype measurement program confirmed the view that the Columbia Estuary, like any other major estuary, is influenced by tidal forces to an extent governed by geography and the complex interaction of these forces with density, littoral, and perhaps other forces significant to the various problems experienced in the estuary. The program definitely identified the normal estuarine characteristic of upstream flow predominance along the bottom levels of the lower estuary in contrast to the downstream predominance noted in the upper levels. This revealed existence of a generally defined pattern of circulation within that region, which has governed the deposition of sediments forming the shoals obstructive to navigation. It has also given rise to the view that the estuary now acts as a vast sediment trap, in which the phenomenon of density currents preclude, except during rare periods of extreme upland discharge, the movement of bottom sediments to the sea.

In consideration of these findings, the Committee on Tidal Hydraulics concluded that while it would be possible to obtain, by extensive and costly analytical analyses of observed data together with collection of additional data, an element of success in determining the cause or causes of the existing shoaling and other problems, such analyses could not accurately predict the effects of improvements works.

Analyses of anticipated future problems would be even more difficult, if not impossible. Also, the effects of regulatory works that are

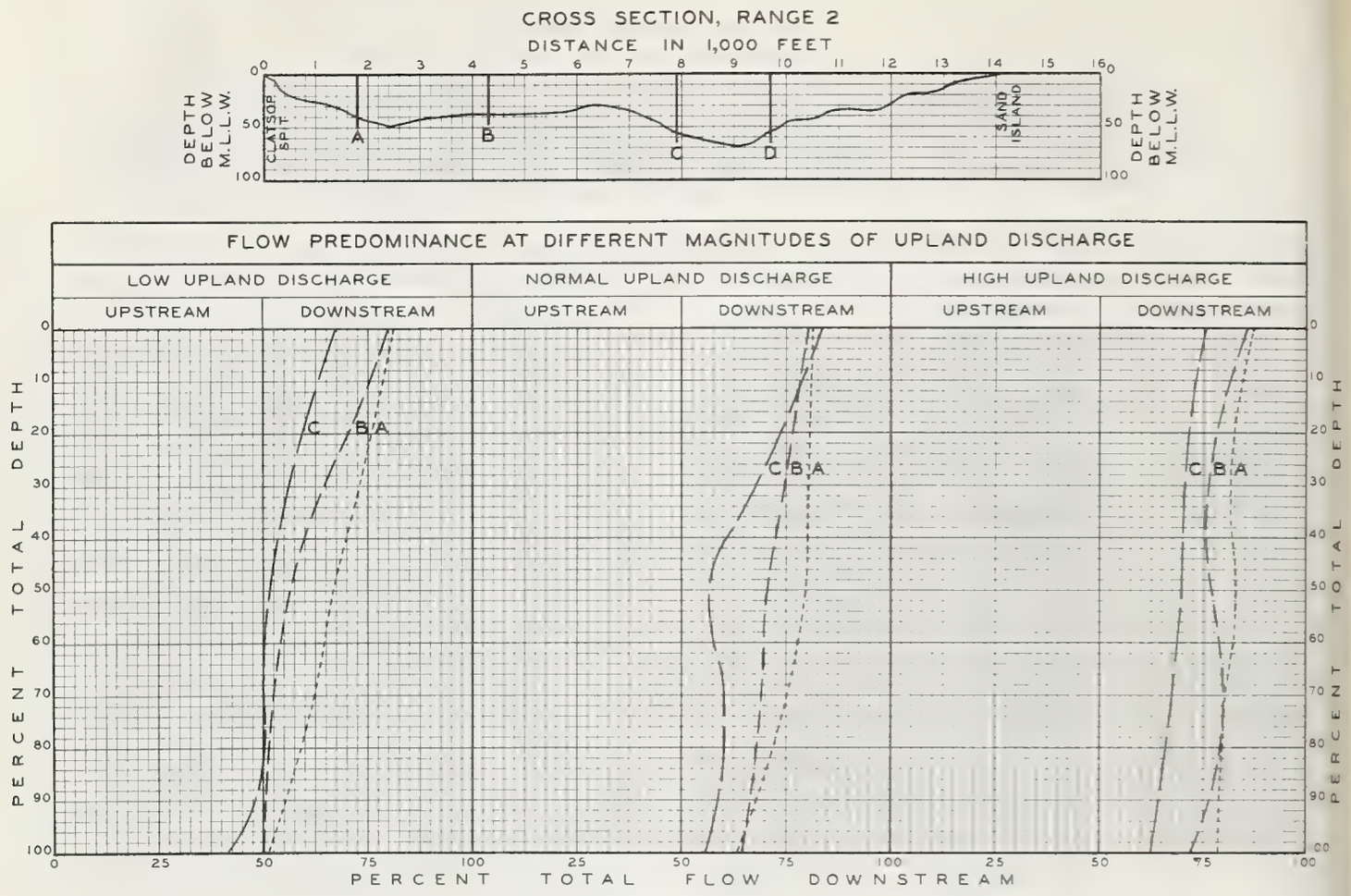


FIGURE 16. — Flow predominance, range 2.

completed, in progress, or planned for the Upper Columbia River and its tributaries will be very significant in the estuarine section of the river.

In the light of all these conditions, the committee felt that more must be learned regarding the physical factors governing the behavior of the waterway and that a comprehensive hydraulic model of the lower estuary represented the most important and urgently needed tool to aid in the development of this knowledge. Accordingly, the committee recommended that immediate steps be taken toward construction of a suitable hydraulic model of the Lower Columbia Estuary from Oak Point to the sea as the initial phase of a comprehensive model investigation.

Offshore and Estuarine Scour and Shoal

Long-Term Trends

Concurrently with the general analysis of observed data, studies were made of the general shoaling patterns since 1945 reflected by condition surveys made of the immediate offshore area, the entrance area, and two shoal areas upstream therefrom (Desdemona and Flavel shoals). These latter shoal areas are located in the vicinities of miles 8.8 and 10.7, respectively. In the review of these particular studies it should be recognized that they show only the changes occurring within the limits of the authorized channel, and, consequently, such studies are not capable of showing the character or magnitude of changes occurring elsewhere throughout the vast estuary and entrance area. Although these studies represent an analysis of known changes, they cannot, due to the limited area covered, reveal more than an indication of the changes occurring in this vast area.

A somewhat clearer indication of the changes occurring in the lower estuary is revealed by a preliminary study of long-term scour and shoal trends made in 1961, which compared bank-to-bank hydrographic surveys of 1868 and 1958 within an 11-mile reach between north and south lines passing through Upper Sand Island and Tongue Point. This study indicated that the total net shoaling occurring within this reach of the estuary, which did not include the

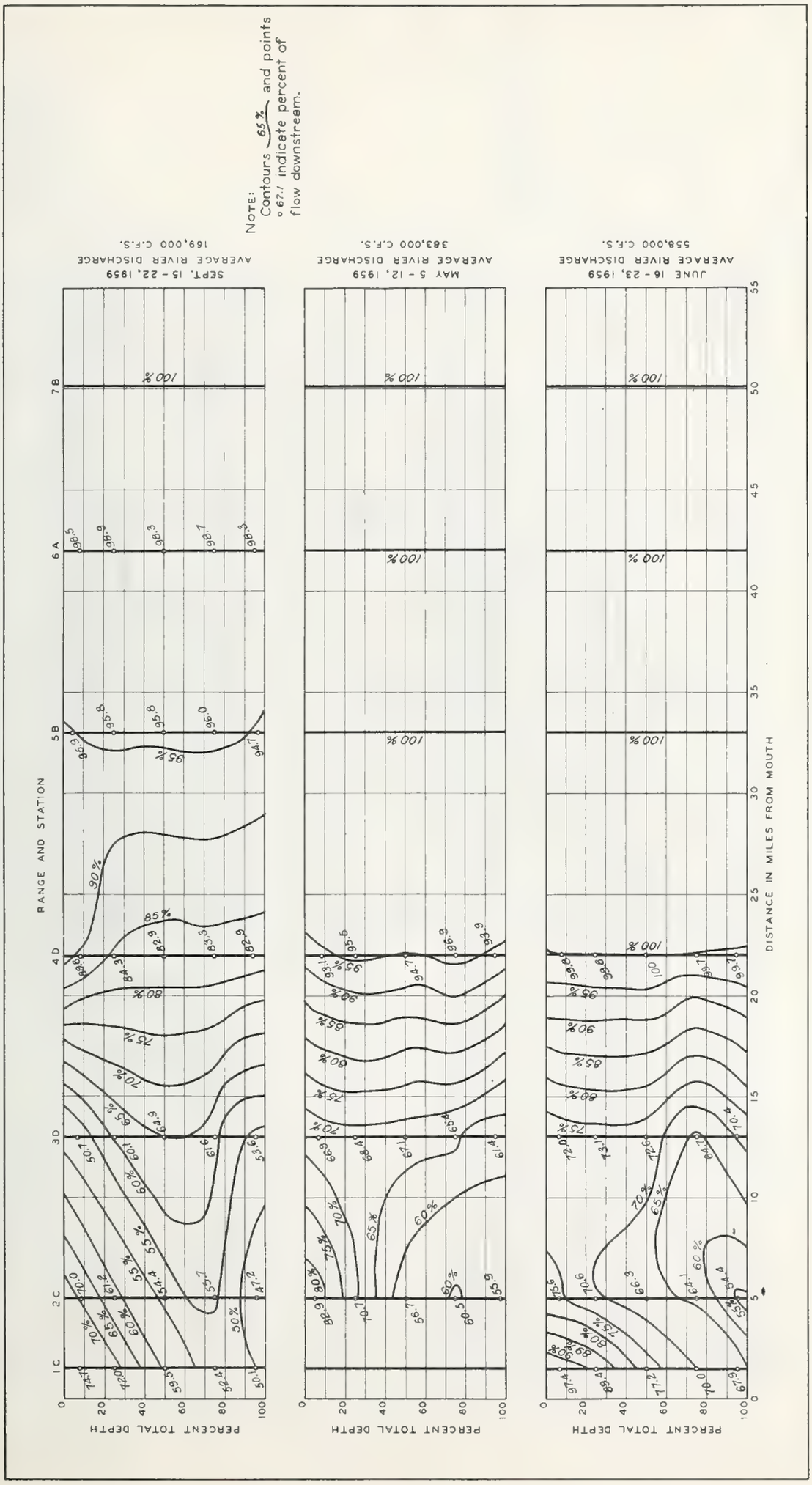


FIGURE 17.— Flow predominance profile, north channel.

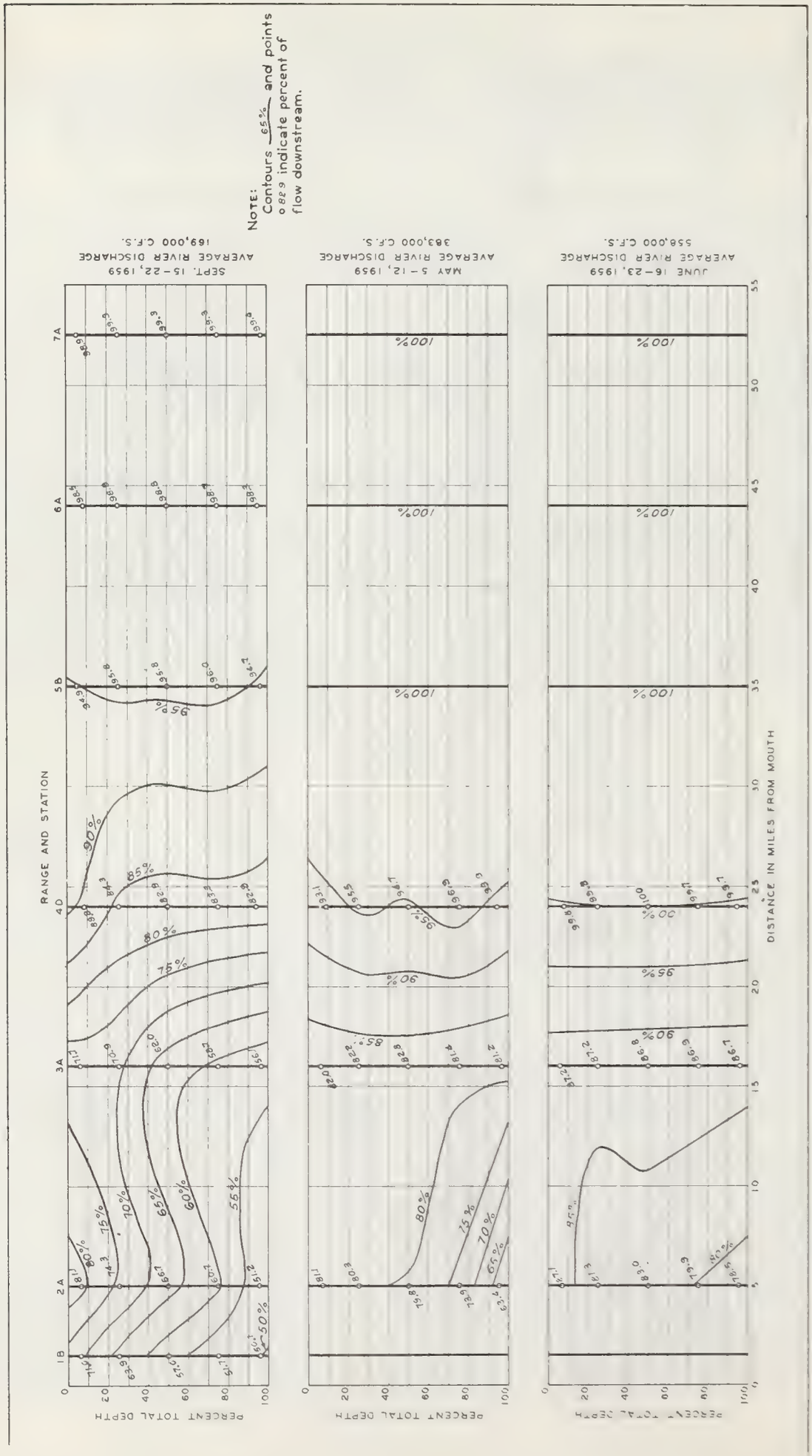


FIGURE 18. — Flow predominance profile ship, channel.

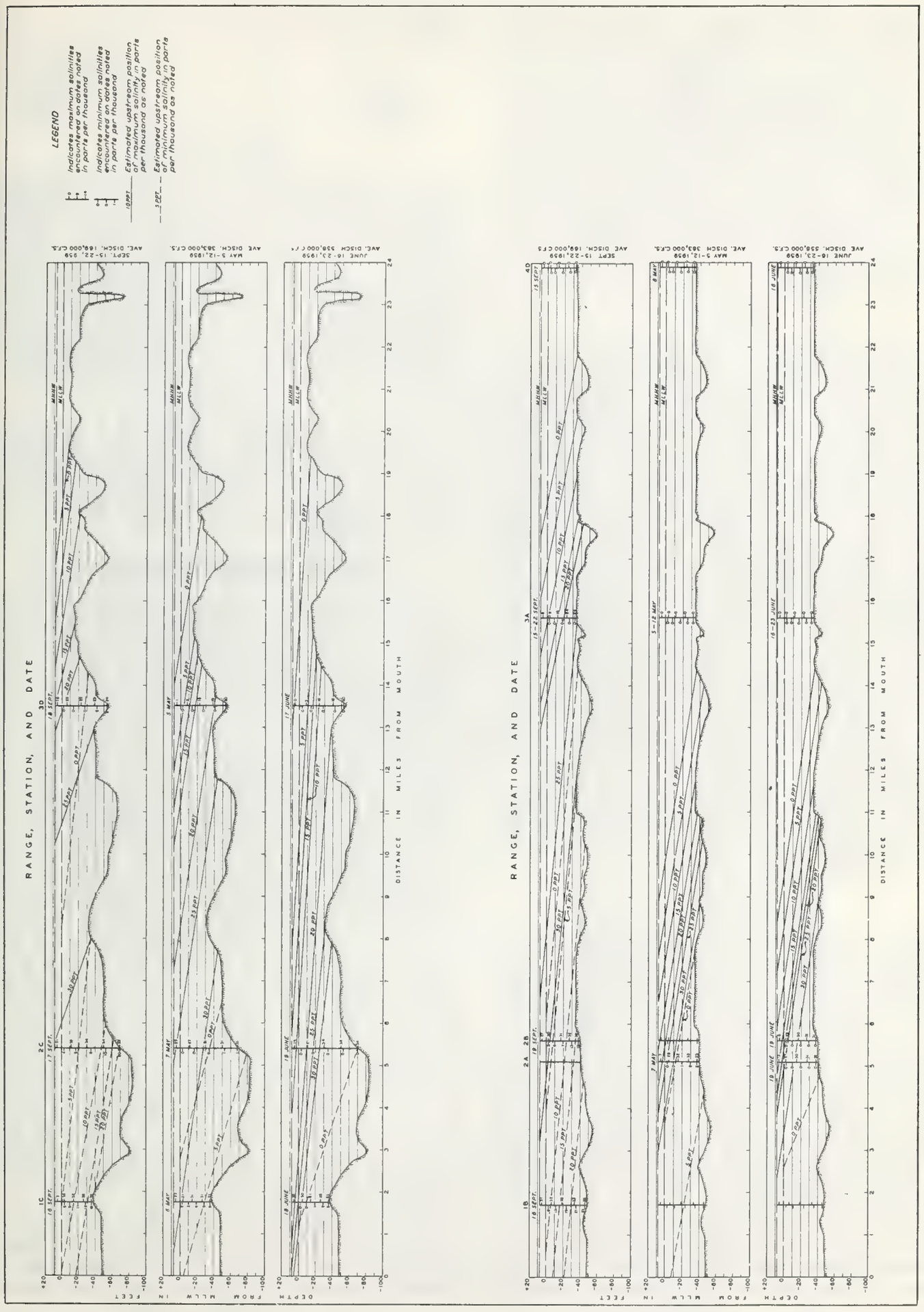

LEGEND
Indicates maximum salinities encountered on dates noted in parts per thousand
Indicates minimum salinities encountered on dates noted in parts per thousand
Estimated upstream position of maximum salinity in parts per thousand as noted
Estimated upstream position of minimum salinity in parts per thousand as noted
RANGE, STATION, AND DATE
DISTANCE IN MILES FROM MOUTH
DEPTH IN FEET FROM MLLW
AVE. DISCH. 169,000 C.F.S.
AVE. DISCH. 383,000 C.F.S.
AVE. DISCH. 338,000 C.F.S.
SEPT. 13-22, 1939
MAY 5-12, 1939
JUNE 16-23, 1939

FIGURE 19.— Salinity profiles.

tremendous Clatsop Shoals, amounted to 77 million cubic yards of material over the 90-year period. Deposition of this amount of material represents an average shoaling of about 3 feet over the 40 square miles of area considered. This preliminary study tends to confirm the view that the Columbia Estuary, since the days of initial improvement, has been acting as a vast sediment trap. Of this amount of shoaling, 66 million cubic yards, or approximately 90 percent occurred in the lower 6-mile reach below Astoria, which includes the Desdemona and Flavel Shoal areas.

Offshore

A study of offshore conditions landward of the 16-fathom depth line from 6 miles south to an equal distance north of the Columbia entrance shows that great changes have occurred in this area since 1877. During the 50-year period, 1877 to 1926, which corresponds roughly to the period of jetty construction, this study revealed that there occurred a net accretion of 183 million cubic yards of material north of the entrance and a net erosion of 374 million cubic yards of material south of the entrance, as shown on figure 20. This change is attributed in large part to the interception by the North Jetty of the predominately southbound littoral drift, which resulted in the accumulation of material north of the entrance and starvation of the offshore area immediately to the south. During the following 32-year period, 1926–58, in which the influence of the jetties was continued, the study revealed that the incremental net accretion to the north of the entrance amounted to about 134 million cubic yards, whereas incremental erosion to the south amounted to about 130 million cubic yards, as shown in figure 21.

Entrance Channel

Studies of changes occurring within the entrance channel since 1945 have revealed a trend for increased shoaling in this channel throughout this period, notwithstanding the change in project depths from 40 to 48 feet effected in 1956, and the associated increased dredging made necessary thereby. Figure 22 shows in graphical form accumulated dredge quantities material remaining in the authorized entrance channel above 42 feet of depth, accumulated scour and shoal, and average rates of shoaling from 1945 to 1955, during which time the 40-foot project was being maintained. Figure 23 shows similar information for the period from 1956 to 1960, during which the 48-foot project was being developed. The average rates of shoaling within the entrance channel during these periods as shown on these graphs are as follows:

Period	*Average rate of shoaling Cubic yards per year*
1945–52	909,000
1953–55	1,518,000
1956-60	1,931,000

Desdemona Shoal

The results of similar studies of changes occurring within the authorized navigation channel, 35 feet deep and 500 feet wide, through Desdemona Shoal during the period 1945–60 are reflected on figure 24. The average rate of shoaling prior to the change in channel alinement, effected in January 1951, amounted to about 458,000 cubic yards a year. Since that time the shoaling rate has been reduced to an average of about 270,000 cubic yards a year.

Flavel Shoal

Figure 25 shows the results of studies made of changes occurring within the authorized navigation channel through Flavel Shoal since 1946. The average shoaling rate in this area during this period has been about 270,000 cubic yards a year.

Attrition South of South Jetty

As previously mentioned, serious attrition has been taking place in adjacent offshore areas along the Oregon coastline immediately south of the South Jetty since the days of initial improvement. Perhaps the most startling visual evidence of this change is the gradual erosion during the last 25 years of the massive sand dunes which protected the arm of land connecting the midpoint of the South Jetty as completed in 1895 with Point Adams. These dunes have been breached along several hundred feet of shore immediately south of the structure and are quite thin for some distance farther south.

The driftwood line indicates that the remaining beach berm has been overtopped during recent storms and it now appears that a complete breach to the lagoon in rear of the shore may occur at any time and is inevitable within a few years unless action is taken to prevent such a catastrophe. The low, permeable enrockment along the remains of the trestle at the root of the jetty separates the lagoon from the estuary proper, as shown in figure 26, but the tide ebbs and flows freely through this structure. Because of the lag in tidal time between this part of the estuary and the open sea, about 40 minutes, a substantial head would develop along this light enrockment in the event of a breach between the sea and the lagoon.

It is quite probable that failure of the structure by scour would be rapid in that eventuality and the breach would quickly develop into the primary course of the river outlet. A program

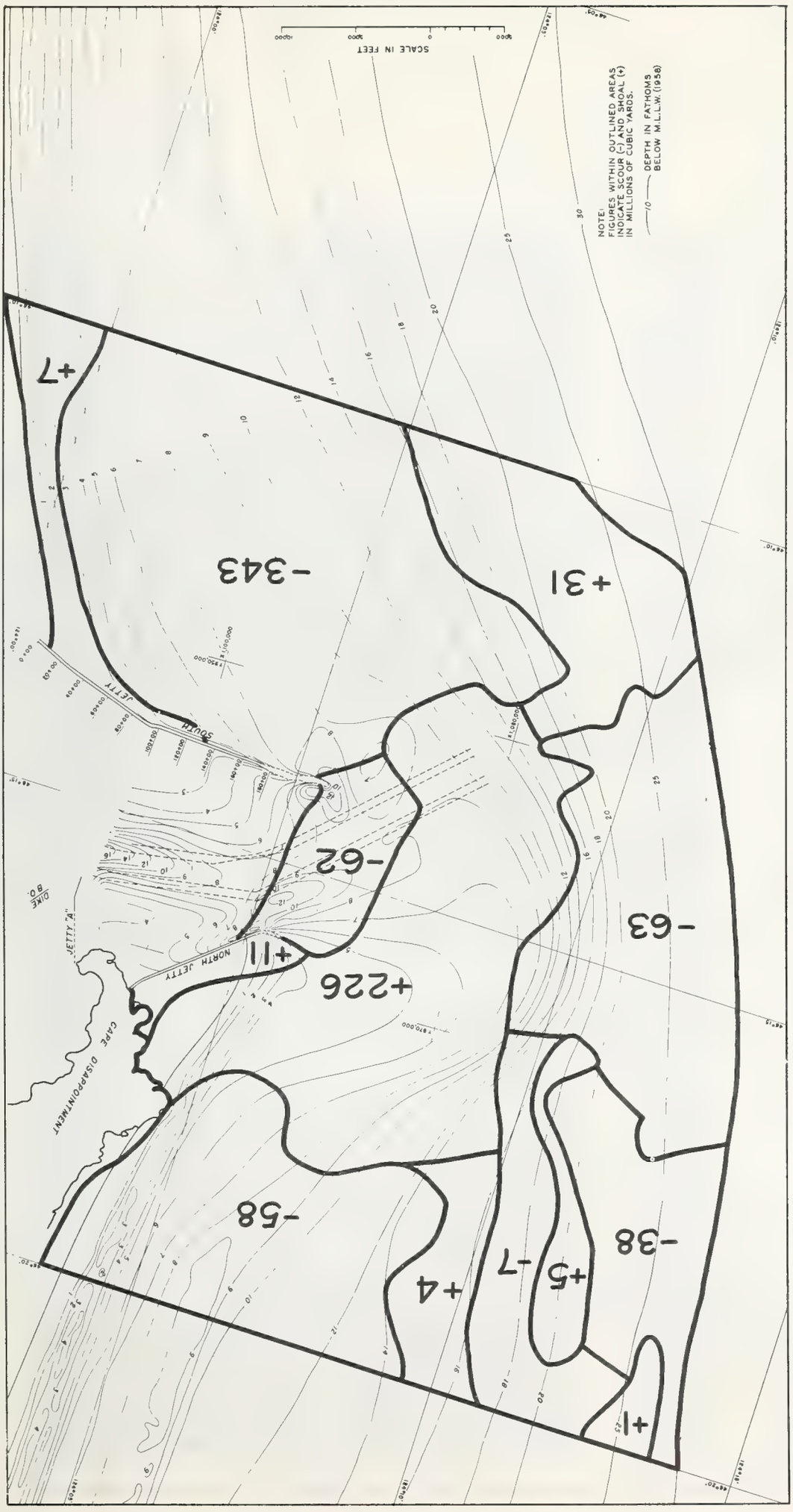


FIGURE 20. — Offshore scour and shoal volumes, 1877-1926.

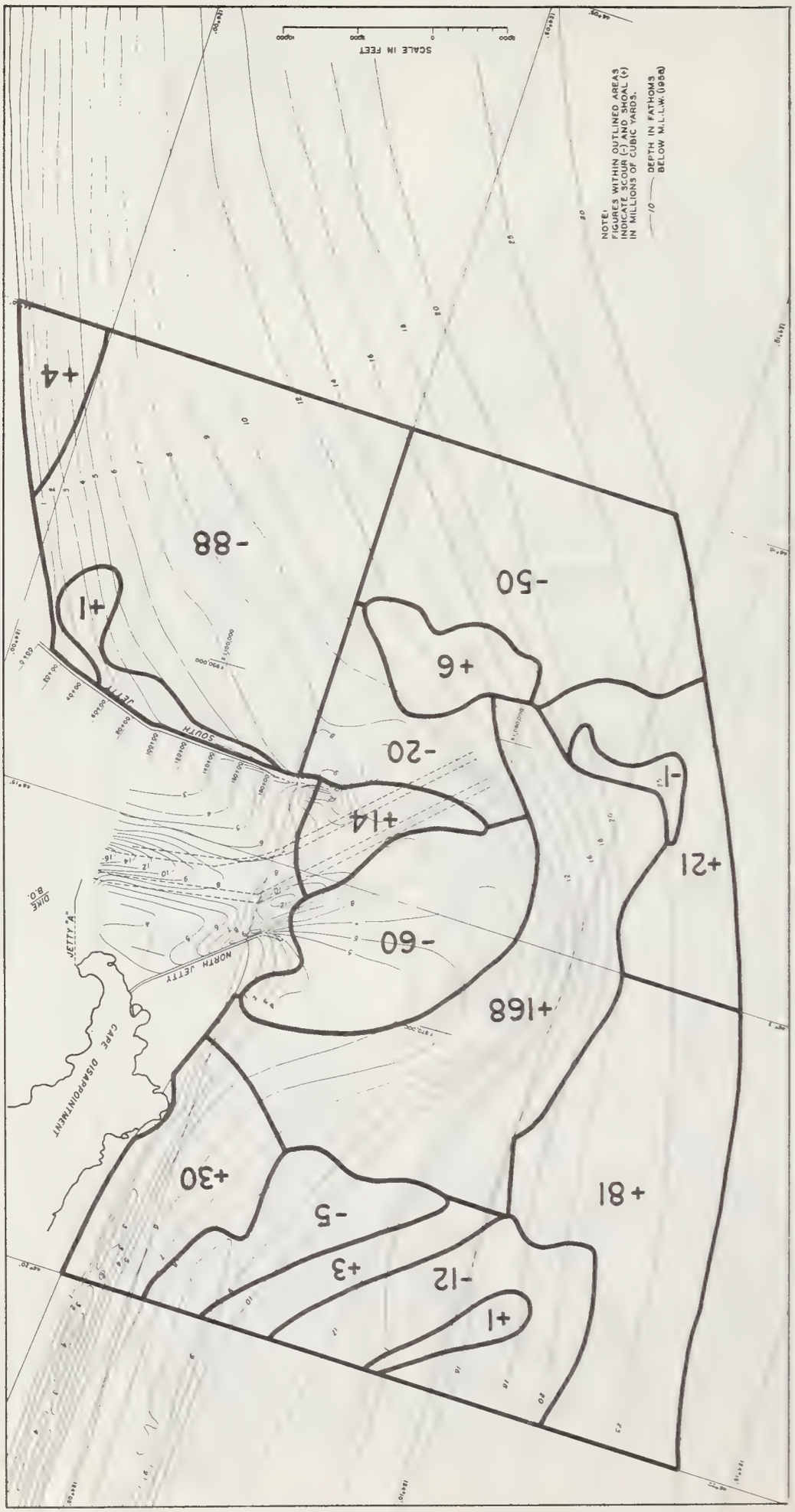


FIGURE 21. — Offshore scour and shoal volumes, 1926-58.

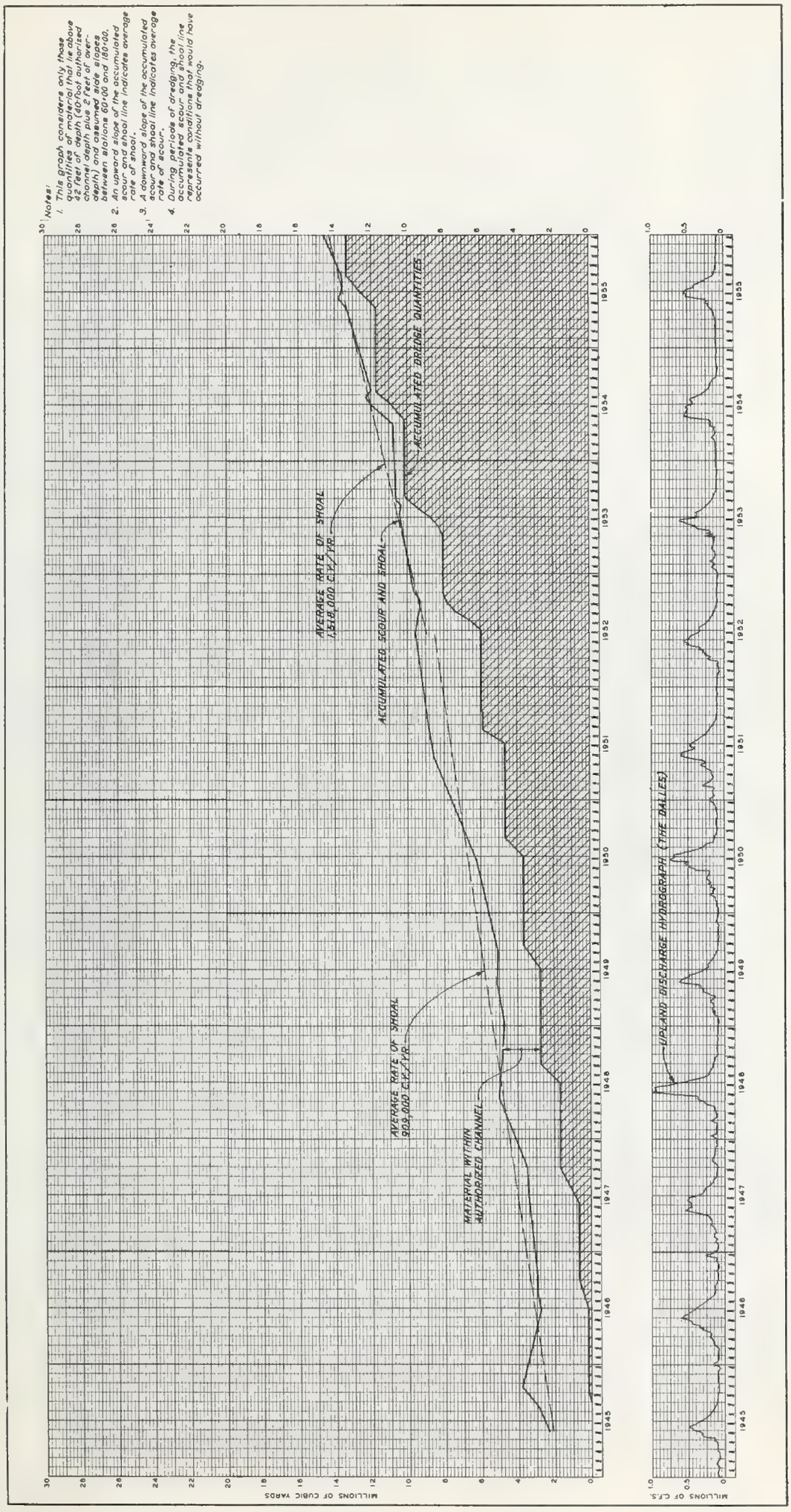

Notes:
1. This graph considers only those quantities of material that lie above 42 feet of depth (40-foot authorized channel depth plus 2 feet of over-depth) and assumed side slopes between stations 60+00 and 180+00.
2. An upward slope of the accumulated scour and shoal line indicates average rate of shoal.
3. A downward slope of the accumulated scour and shoal line indicates average rate of scour.
4. During periods of dredging, the accumulated scour and shoal line represents conditions that would have occurred without dredging.
AVERAGE RATE OF SHOAL 1,516,000 CU.YD./YR.
ACCUMULATED SCOUR AND SHOAL
ACCUMULATED DREDGE QUANTITIES
AVERAGE RATE OF SHOAL 909,000 CU.YD./YR.
MATERIAL WITHIN AUTHORIZED CHANNEL
MILLIONS OF CUBIC YARDS
UPLAND DISCHARGE HYDROGRAPH (THE DALLES)
MILLIONS OF C.F.S.

FIGURE 22.—Scour and shoal, 40-foot channel.

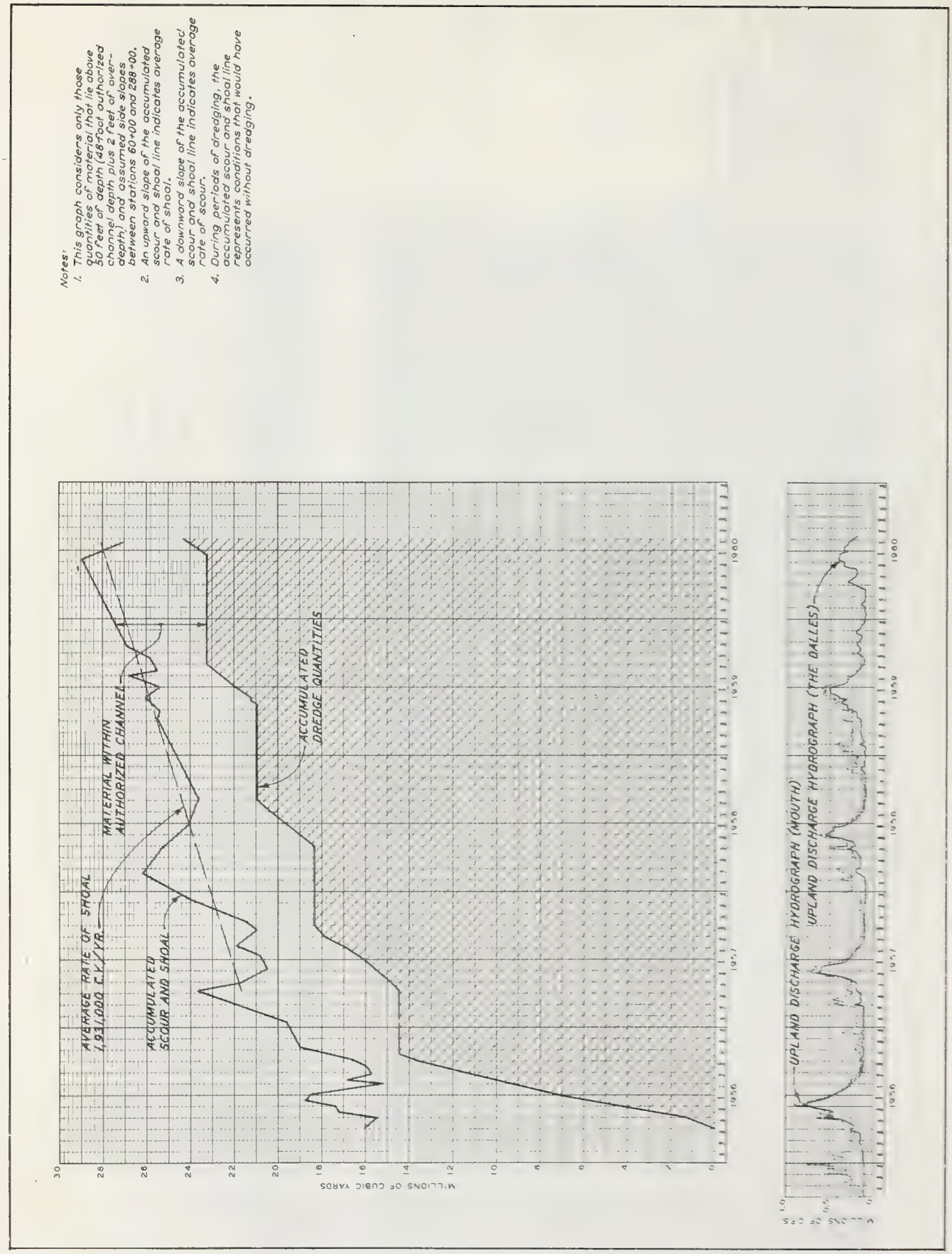

Notes:
1. This graph considers only those quantities of material that lie above 50 feet of depth (48-foot authorized channel depth plus 2 feet of over-depth) and assumed side slopes between stations 60+00 and 288+00.
2. An upward slope of the accumulated scour and shoal line indicates average rate of shoal.
3. A downward slope of the accumulated scour and shoal line indicates average rate of scour.
4. During periods of dredging, the accumulated scour and shoal line represents conditions that would have occurred without dredging.
AVERAGE RATE OF SHOAL 1,931,000 C.Y./YR.
MATERIAL WITHIN AUTHORIZED CHANNEL
ACCUMULATED SCOUR AND SHOAL
ACCUMULATED DREDGE QUANTITIES
MILLIONS OF CUBIC YARDS
UPLAND DISCHARGE HYDROGRAPH (MOUTH)
UPLAND DISCHARGE HYDROGRAPH (THE DALLES)
1956
1957
1958
1959
1960

FIGURE 23. — Scour and shoal, 48-foot channel.

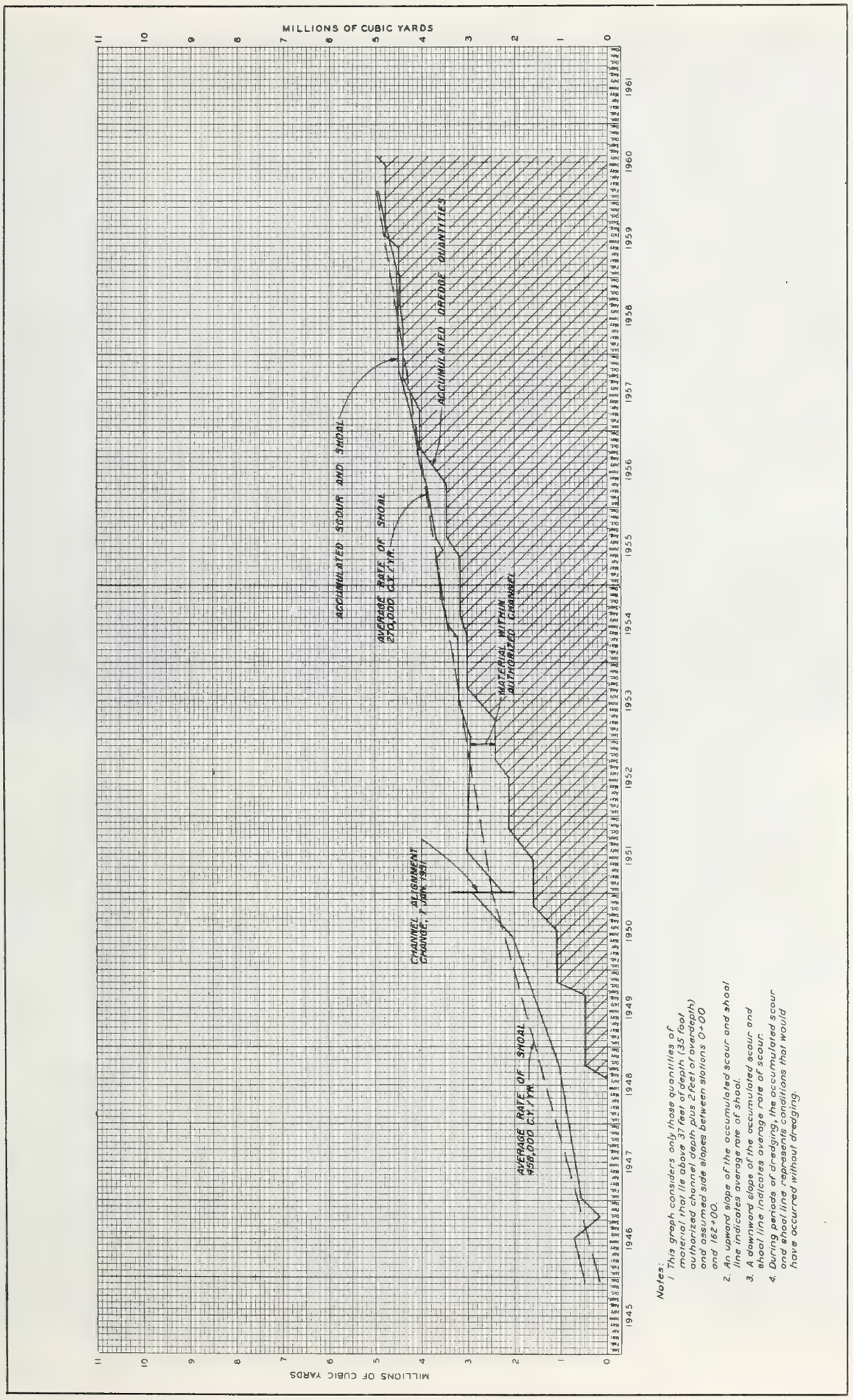

MILLIONS OF CUBIC YARDS
ACCUMULATED SCOUR AND SHOAL
AVERAGE RATE OF SHOAL 270,000 CY/YR.
ACCUMULATED DREDGE QUANTITIES
MATERIAL WITHIN AUTHORIZED CHANNEL
CHANNEL ALIGNMENT CHANGE, 1 JAN 1951
AVERAGE RATE OF SHOAL 458,000 CY/YR.
1945
1946
1947
1948
1949
1950
1951
1952
1953
1954
1955
1956
1957
1958
1959
1960
1961
Notes:
1. This graph considers only those quantities of material that lie above 37 feet of depth (35 foot authorized channel depth plus 2 feet of overdepth) and assumed side slopes between stations 0+00 and 162+00.
2. An upward slope of the accumulated scour and shoal line indicates average rate of shoal.
3. A downward slope of the accumulated scour and shoal line indicates average rate of scour.
4. During periods of dredging, the accumulated scour and shoal line represents conditions that would have occurred without dredging.

FIGURE 24. — Scour and shoal, Desdemona shoal.

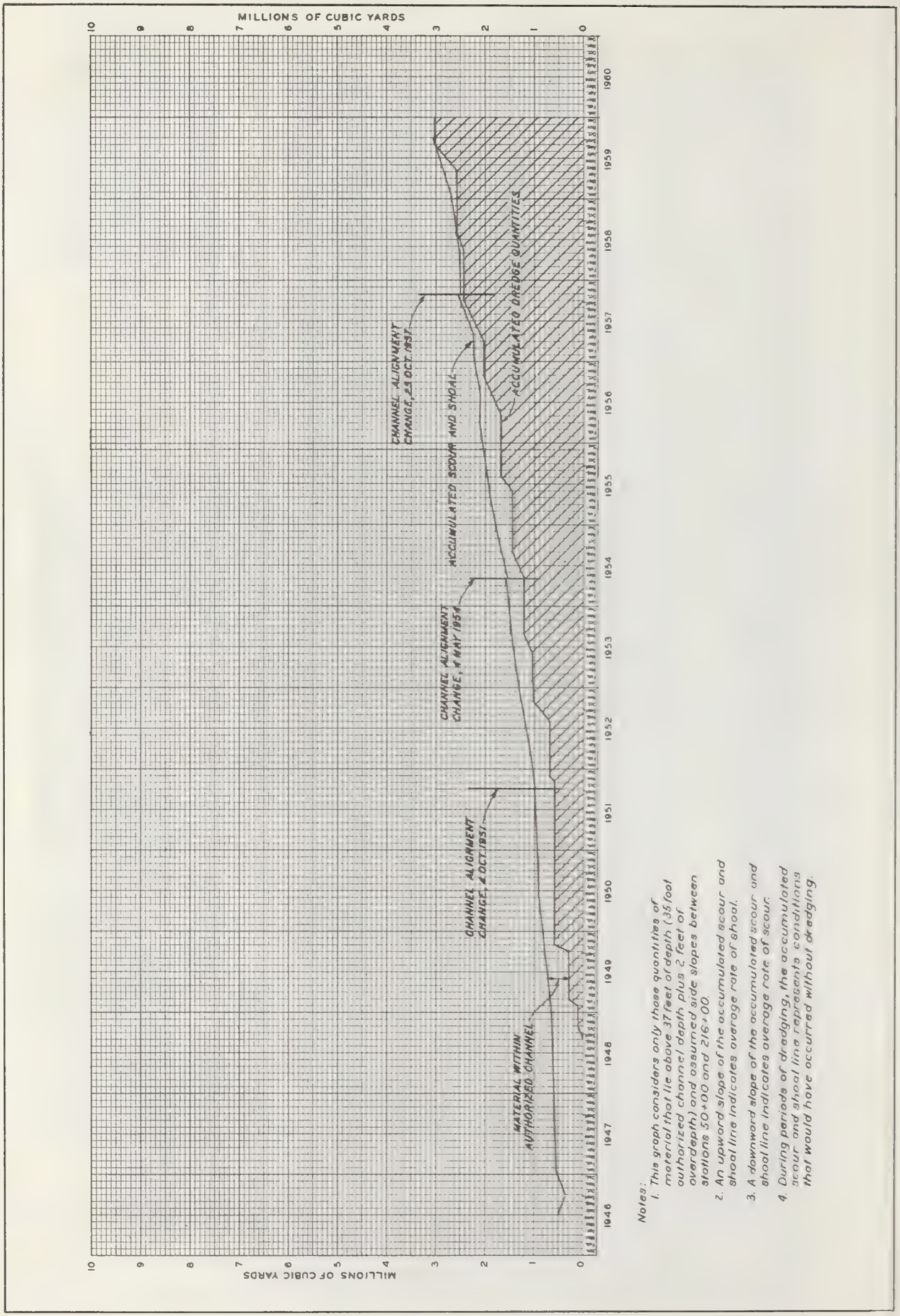


FIGURE 25. — Scour and shoal, Flavel shoal.

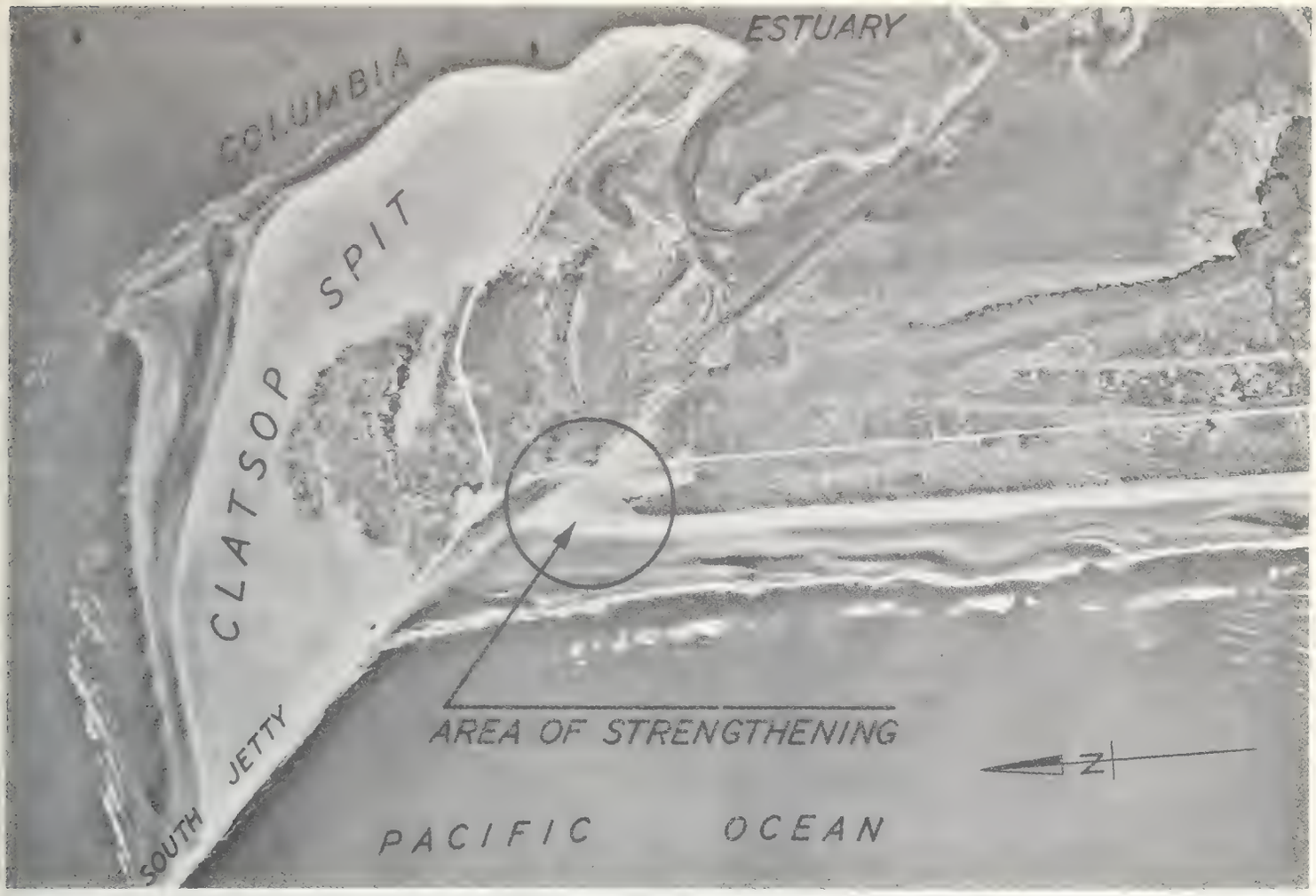


FIGURE 26. — Erosion south of South Jetty.

of study and surveillance of offshore conditions in the vicinity of the South Jetty has been launched in an effort to determine the cause or causes of this problem with the view to undertaking appropriate corrective action. In the meantime, a temporary structural strengthening of the threatened area has been made to prevent a relocation of the entrance channel, pending completion of the study and surveillance program and analysis of the problem in the hydraulic model hereinafter described in detail.

Satistical Wave Studies

In continuation of Corps of Engineers' studies of characteristics of waves off the California coast, a contract was awarded in August 1960 to the National Marine Consultants, Inc., Santa Barbara, Calif., for accomplishment of studies, leading to the development of characteristics of waves prevailing off the coasts of Oregon and Washington. These particular studies had as their objectives: (1) the development of wave statistics for three deep-water stations off the Oregon and Washington coasts based on conditions prevailing in 1956, 1957, and 1958; (2) statistics for 12 most severe storms at these selected stations during the period from 1950 to 1960; and (3) structural design wave analyses and wave modification analyses of conditions prevailing at and near the Columbia Entrance during the period 1940–60. The results of these studies, discussed in the following paragraphs, are contained in manuscript reports prepared by National Marine Consultants, Inc.[8]

Hindcast Studies, 1956, 1957, and 1958

The general area covered by these studies embraced the entire Pacific coast of Oregon and Washington as represented by three selected deep-water stations: one located off the entrance to Yaquina Bay, one off the Columbia Entrance, and one off the coast between Grays Harbor

[8] NATIONAL MARINE CONSULTANTS, INC. WAVE STATISTICS FOR TWELVE MOST SEVERE STORMS AFFECTING THREE SELECTED STATIONS OFF THE COASTS OF WASHINGTON AND OREGON, DURING THE PERIOD 1950-1960. January 1961; OCEANOGRAPHIC STUDY FOR COLUMBIA RIVER ENTRANCE. March 1961; WAVE STATISTICS FOR THREE DEEP WATER STATIONS ALONG THE OREGON-WASHINGTON COAST. May 1961. Santa Barbara, Calif.

and the Strait of Juan De Fuca. On the basis of 6-hour U.S. Weather Bureau synoptic charts covering extratropical cyclones and the Pacific high pressure cell, these studies routed the paths and decay of meteorological disturbances to develop the height, direction, and period of the resulting wave conditions at the deep-water stations. The extratropical cyclones considered in these studies included storms moving into the Gulf of Alaska, storms moving directly into the study area, storms moving onto the California coast, and western Pacific storms.

In addition to development of the characteristics of waves resulting from these meteorological conditions termed "swell," the studies also determined similar characteristics for waves caused by local wind conditions termed "sea." This latter condition often constituted a separate source of swell at an adjacent station, inasmuch as a wave in the form of "sea" at one station would show up as "swell" at an adjacent station not under the influence of the same wind condition. It was frequently found during the course of the studies that as many as six separate wave trains would sometimes occur simultaneously at the deep-water stations. As a consequence, in the annual summary shown in table 5, the time over which "swell" conditions occur is greater than the elapsed time over the period covered. This tabulation and table 6 show the significant wave height and direction frequency of waves caused by "swell" and "sea," respectively, at the deep-water station off the Columbia Entrance, located at latitude 46° 12′ North and longitude 124° 30′ West:

TABLE 5.—*"Swell": Average annual height and direction [1] frequency distribution [2]*

Average total hours: 10,508.45

Height of "swell" (feet)	NNW	NW	WNW	W	WSW	SW	SSW	S	Total Σ
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1 to 2.9	0.68	8.20	12.67	13.69	6.67	2.31	2.85	1.90	48.97
3 to 4.9	.04	2.16	12.36	13.61	4.67	1.83	1.88	.81	37.36
5 to 6.9	.04	1.18	6.14	5.51	1.37	.98	1.07	.39	16.68
7 to 8.9	.04	.45	2.50	2.67	.54	.74	.55	.23	7.72
9 to 10.9	.05	.20	1.11	1.43	.47	.36	.30	.16	4.08
11 to 12.9	.....	.09	.66	.63	.30	.09	.11	.13	2.01
13 to 14.9	.....	.02	.28	.61	.21	.06	.12	.04	1.34
15 to 16.9	.....	.02	.13	.21	.09	.04	.13	.10	.72
17 to 18.9	.....	.07	.06	.20	.06	.02	.02	.02	.45
19 to 20.9	.....	.04	.04	.07	.02	.....	.....	.....	.17
21 to 22.9	.....	.....	.02	.09	.....	.....	.....	.....	.11
23 to 24.9	.....	.....	.....	.04	.07	.....	.....	.....	.11
25 to 26.9	.....	.....	.....	.....	.....	.....	.....	.....	.....
27+	.....	.....	.....	[3] .04	.....	.....	.....	.....	.04
Total Σ	0.85	12.43	35.97	38.80	14.47	6.43	7.03	3.78	119.76

[1] Direction from which waves approach the deep-water station.
[2] Based on 365⅓ days.
[3] 30 feet (March 1956).

TABLE 6.—*"Sea": Average annual height and direction[1] frequency distribution[2]*

Height of "sea" (feet)	N	NNW	NW	WNW	W	WSW	SW	SSW	S	Total Σ
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1 to 2.9	2.06	2.25	7.32	1.44	2.77	1.00	2.49	1.57	3.07	23.97
3 to 4.9	.84	1.76	5.25	1.45	1.51	1.21	1.61	1.40	2.43	17.46
5 to 6.9	.42	.62	1.71	.54	.79	.73	1.11	1.44	1.85	9.21
7 to 8.9	.16	.28	1.22	.53	.60	.46	1.14	1.04	1.06	6.49
9 to 10.9	.02	.....	.37	.21	.16	.14	.23	.63	.80	2.56
11 to 12.9	.....	.04	.43	.13	.09	.11	.35	.45	.95	2.55
13 to 14.9	.....	.07	.05	.09	.09	.10	.23	.45	.45	1.53
15 to 16.9	.....	.....	.05	.....	.09	.....	.....	.12	.25	.51
17 to 18.9	.....	.02	.07	.02	.....	.02	.05	.19	.35	.72
19 to 22.9	.....	.....	.....	.02	.....	.....	.....	.12	.06	.20
23 to 27	.....	.....	.....	.....	.....	.....	.04	.02	.04	.10
Total Σ	3.50	5.04	16.47	4.43	6.10	3.77	7.25	7.43	11.31	100.00

[1] Direction from which waves approach the deep-water station.
[2] Based on 365⅓ days.
[3] Frequency distribution of "sea" offshore of 6.03 percent and of calm of 28.67 percent; calm includes waves of 0 to 0.9 foot.

Severe Storm Wave Characteristics, 1950-60

Generally, extratropical cyclones giving rise to severe wind and wave conditions along the Oregon-Washington coast originate near Japan and move eastward across the Pacific toward the Gulf of Alaska. The degree to which these disturbances affect conditions off the Columbia Entrance depends not only upon the intensity of the storm but also on the disposition of the Pacific high cell. During the summer months this cell is sufficiently removed to the north so that only a few storms approach the Oregon-Washington coast and those that do are relatively weak.

During the period from October to May, however, with the southward shift of the Pacific cell, intense storms approach the coast. These storms develop in the form of rapidly moving intense frontal systems on low pressure centers and occur several times each winter. Although prefrontal southerly winds often attain velocities of over 100 miles per hour in gusts along the coast, the winds associated with these storms seldom reach hurricane force over much of the area. However, the extent of storms in terms of duration and fetch can produce hurricane magnitude waves that last for relatively long periods of time.

Principal wind fields associated with these storms produce south to southeasterly winds preceding a warm front, southwest to west-southwest winds in the warm sector, and west-southwest to northwest winds behind a cold front. Each of these wind regimes can develop high waves, but offshore topography and coastal promontories dictate shallow water effects at coastal locations. Hindcast methods reveal the characteristics of severe storm waves in deep water off the Columbia Entrance during the period 1950–60 as shown in table 7.

TABLE 7.—*Severe storm wave characteristics*

Date	Significant height	Significant period	Direction from—
	Feet	*Seconds*	
27 Oct. 1950	24	11	SW
21 Jan. 1951	27	14	WSW
2 Feb. 1951	25	13	WSW
18–19 Dec. 1951	23	12	NW
6–7 Dec. 1952	25	11	SW
5 Dec. 1953	24	14	WSW
12–13 Feb. 1954	30	13	SSW
19 Nov. 1954	23	12	SW
15 Jan. 1956	24	12	SW
3 Mar. 1956	30	12	W
26 Dec. 1957	23	13	W
23 Jan 1958	26	11	SSW
11 Dec. 1959	25	13	SW

Structural Criteria of Columbia Entrance

This study was composed of two parts — one part considered the characteristics of storm waves as applied to the design of structures at the Columbia Entrance and the other part related to refraction patterns assumed by waves from different directions and of different periods as they approach a 20-mile sector of the coast centered on the Columbia Entrance. The structural design wave analyses were based on hindcast studies of a number of severe storms that approached the entrance during the last 20 years. Considering the deep-water characteristics of waves generated by the previously mentioned severe storms and the physical factors tending to modify these waves, such as exposure, refraction, shoaling, bottom friction and percolation, bottom slope and still-water depth at the entrance site, as well as the frequency distribution of wave heights, it was determined that structural criteria at the Columbia entrance should be based on the following general wave conditions:

Significant wave height

(H_s) ..... 30.0^{+4}_{-2} feet

Significant wave period

(T_s) ..... 13.0^{+1}_{-1} seconds

Direction of approach (θ) $235°^{+20}_{-15}$ (from)

With a tide range of 8.5 feet, storm setup of 2.5 feet, and design depth of 51 feet, the most severe condition occurring within the period of study produced a wave with a significant height of 33.5 feet, significant period of 13 seconds, and an angle of approach of 234° as shown in table 8.

TABLE 8.—*Design wave characteristics*

Deep water			Modification		Design wave at entrance		
H_s (feet)	T_s (sec.)	Θ (dir.)	Refraction coefficient K_r	Shoaling coefficient K_s	H_s (feet)	T_s (sec.)	Θ (dis.)
30	13	205°	0.94	0.996	28.0	13	206°
25	13	225°	1.11	.996	27.5	13	218°
28	13	245°	1.20	.996	33.5	13	234°
30	13	270°	.97	.996	29.0	13	254°
28	13	295°	.71	.996	20.0	13	263°
23	12	315°	1.24	.996	27.5	13	278°

Radioactive Analysis of Bottom Samples

In connection with the broad study being conducted by the Atomic Energy Commission of the distribution of Columbia River sediments as related to discharge of radioactive waste effluent from the Hanford complex, which is subsequently discussed, samples of bottom materials were obtained from five different points within the Columbia Entrance and from one location in the lower Estuary. In the Entrance area, one (sample A) was obtained from the channel on Range 2, two (samples B and C)

were obtained from each side of the entrance channel on Range 1, one (sample D) in the entrance channel between the ends of the jetties, and one (sample E) in the channel about 1 mile outside the jetties. In the lower estuary one sample (sample F) was obtained from the channel near Harrington Point.

These samples were analyzed by the General Electric Co. in Hanford to ascertain the relative ages of isotopes present in an endeavor to determine the source of materials forming the samples. In these samples, the radioisotope concentrations were extremely low in comparison with the natural radioactive content of the material. However, it was possible to make age estimates based on the Zn^{65} (245 day half-life) and Cr^{51} (26 day half-life) concentrations. A high Cr^{51} concentration relative to the Zn^{65} concentration would represent a "new" material whereas a low Cr^{51} concentration relative to Zn^{65} would represent much "older" material. The observed Zn^{65} and Cr^{51} concentrations in the above samples and their calculated ratio and relative ages were determined as shown in table 9. The

TABLE 9.—*Analysis of samples*

Sample	Concentration D/M/gram		Ratio $\frac{Cr^{51}}{Zn^{65}}$	Relative age
	Zn^{65}	Cr^{51}		
A	17.1	11	0.64	New.
B	3.9	1	.3	Old.
C	5.8	5	.85	New.
D	3.2	1	.3	Old.
E	6.3	5	.79	New.
F	7.6	30	3.95	Very new.

newer sediments suggest the likelihood that the material has been recently formed of particles moved into the estuary from upland areas, whereas the older sediments very likely have moved into the entrance area from the ocean.

Distribution of Columbia River Sediments

The Atomic Energy Commission has launched a broad program of study and investigation aimed (1) to explore Columbia River sediments as they bear on the fate of radioisotopes released from the Hanford production complex and (2) to determine the interactions taking place in the Columbia Estuary related to the ultimate distribution of sediments as measured by the concentration of radioactivity on these sediments. In this study, it is proposed to collect and analyze samples of bottom and suspended sediment to determine the principal chemical and physical exchange reactions involved, the sedimentation rates and accumulations, and the character of transport downstream of the sediments of sorbed or precipitated isotopes, with the view to depicting in a quantitative manner the radioactive material balance in regard to its interactions with the sediments.

In the estuarine section of the river, the study is directed toward measurement of the physical forces of tides and currents and to sample the estuarine waters, biota, and sediments to determine:

1. The dispersion patterns and rates of interchange of the dissolved radioactivity in the fresh waters with those of the saline, brackish waters;

2. The removal of radioactivity from solution by means of chemical exchange with suspended and bottom sediments in the estuary; and

3. The rate and degree of removal of radioactivity from solution into the biological food web.

Status of Investigation

It is not difficult to visualize that a study of this broad scope and complexity, considering its pioneering nature, will require several years to complete. Nevertheless, considerable progress has already been made along several broad phases of the investigation. Pursuant to contracts made with the Atomic Energy Commission, the Department of Oceanography, University of Washington, has conducted field investigations to determine the characteristics of movement of Columbia River waters in the Pacific Ocean and has traced the fresh water plume of flow for many miles seaward.

Other field investigations by that Department are directed at measurements of ocean salinity, chemical composition, transparency, temperature, density, plankton, and analysis of bottom sediments along the coast, across the continental shelf, and into the abyssal plain of the ocean. In addition, studies by the Laboratory of Radiation Biology of that University of the radioactivity in biological organsms are underway.

Through another contract, the U.S. Geological Survey has activated a program to study fluvial sediment transport in the river as may be related to the sorption of radionucleides associated with such movement. Although this program will initially investigate sediment phenomena at a number of ranges across the Columbia River above Portland, it is possible that this program will be later extended to cover reaches of the river below that point. It is anticipated that the comprehensive hydraulic model of the lower Columbia Estuary and Entrance, subsequently described, will eventually be of much value in revealing the distribution of sediments in these regions.

Related Studies

The Department of Oceanography, Oregon State University, for a number of years has

been engaged in the study of coastal and ocean phenomena prevailing along the coast of Oregon. In addition to its investigation of the nature of sediments in estuarine waters along this coast, the Department is engaged in preparing bathymetric charts of the offshore areas and plans shortly to map and study the physical features of the Astoria Canyon lying off the Columbia Entrance. Its study of the distribution of Columbia River water has revealed the seasonal offshore movements of ocean currents to the south during the summer and to the north during the winter. Offshore cruises have revealed large lenses of traceable river water upwards to 50 miles in diameter and 100 feet thick located hundreds of miles at sea. These lenses of water are somewhat warmer than the river water flowing from the Columbia Entrance.

Comprehensive Hydraulic Model Investigation

Authorization

Pursuant to the previously mentioned recommendations of the Committee on Tidal Hydraulics, which received the concurrence of the district engineer, Portland, and division engineer, North Pacific Division, Portland, Oreg., the chief of engineers in March 1961 authorized the undertaking of a compreshensive hydraulic model investigation of the lower Columbia Estuary and Entrance. As the result, the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Miss., has constructed and will shortly place into operation a hydraulic model that will reproduce the lower 52 miles of the Columbia Estuary and a part of the Pacific Ocean extending about 8 miles north and south of the river entrance and offshore to the 20-fathom contour.

Design Features

The hydraulic model has been constructed to linear scale ratios, model to prototype, of 1:500 horizontally and 1:100 vertically. Tides and tidal currents are being reproduced by a primary tide generator located in the ocean part of the model and a secondary tide generator located at the upstream limit of the model at Oak Point, Wash. The model ocean is equipped with wave generators to reproduce ocean wave effects on movement and deposition of sediments, as well as a circulating system to reproduce the effects of alongshore littoral currents on sedimentation phenomena.

The model will be operated with salt water, salinity scale 1:1, in the model ocean, and with fresh water supplied by the Columbia River and its significant tributaries, so that density effects on hydraulic and shoaling phenomena will be reproduced. The model will be of a combination fixed-bed and movable-bed type, so that critical parts of the model estuary and entrance can be converted from one type to another, thus employing the advantages of each type in conducting studies of the various problems involved. Figure 27 shows the general layout of the model as well as the limits of the fixed-bed and movable-bed parts.

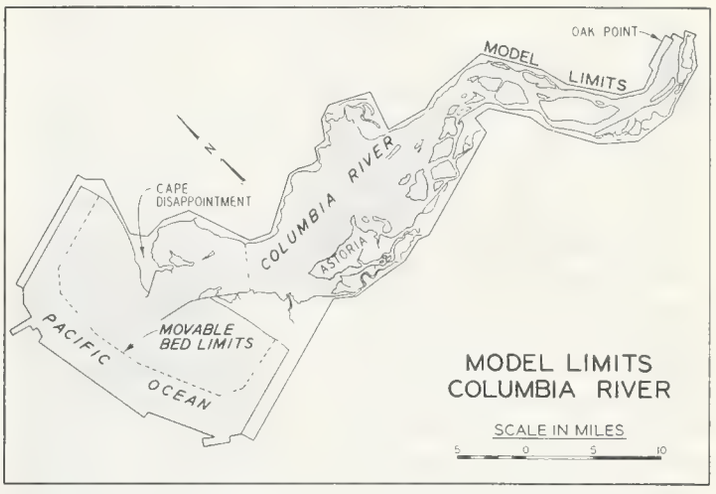


FIGURE 27. — General layout of Columbia hydraulic model.

Model Verification

Hydraulic and salinity verifications of the model were based on prototype measurements of tides, current velocities, current directions, and salinities observed during the 1959 prototype measurement program. The observation stations manned during that program, as well as the range of conditions covered, were specifically planned to provide the data needed for these phases of model verification. As the model is adjusted to reproduce simultaneously all pertinent phenomena observed at the prototype stations during the 1959 measurement program, pertinent phenomena at intermediate points in the prototype are also being reproduced to scale.

Verification of bed movement in the movable-bed part of the model involves an empirical process of adjusting the forces involved, the nature of the bed material, and the duration of model operation until the model will reproduce within sufficient accuracy the significant changes in bed conditions that have occurred in the prototype during known periods of time. These significant changes include the formation of shoals in the navigation channel, accretion and erosion, or erosion in areas outside the navigation channel, and accretion or erosion attributable to structures constructed in the

prototype during the period of time used for model verification purposes.

Once the model has demonstrated its ability to reproduce all significant changes in bed conditions of the prototype, the actual model operation time required to effect such changes is used as a measure of the time scale for bed movement. Thus, if the model consistently shows that changes which occurred in the prototype between hydrographic surveys made at intervals of 1 year are being reproduced in the model during 20 hours of operation, then the time scale for bed movement would be 20 hours model to 1 year prototype, or about 1 to 438.

Although the time scale for bed movement in the Columbia model will not be known until the movable-bed verification is accomplished, the time scale for reproduction of hydraulic and salinity phenomena, derived mathematically from model linear scales, will be about 1 to 50. Thus the model, within the time scales thus determined, will reproduce tidal elevations and phases, current velocities from bank to bank and from surface to bottom, salinity concentrations and distribution throughout the saline part of the estuary, and movements of sediments within the movable-bed part of the model. In addition to reproducing these phenomena, the model will automatically reproduce the dilution, dispersion, and flushing patterns of any contaminants that might be introduced and readily mixed with the waters of the estuarine system.

Model Testing

It is anticipated that during the next 25 years, continued growth and industrial development within the Columbia River basin, including construction and completion of many water development and conservation projects, will have a marked effect on the quality as well as characteristics of Columbia River flows. In order that the Columbia model may be a useful tool during this critical phase of development, the model has been constructed to have a useful life of at least 25 years. During the early part of its life the model will serve to solve problems of present-day urgency, after which it will be available to lend guidance to the solution of future estuarine problems as they develop and affect the welfare of the lower Columbia Estuary and Entrance area. Having been successfully verified, tests now proposed for the model are outlined in the following paragraphs.

Entrance relocation

The threat of a channel relocation through the shore end of the South Jetty resulting from the loss of shore material in that area poses an extremely difficult and challenging problem to those charged with maintenance of a usable and satisfactory entrance channel for navigation. When the enormous buildup of materials immediately offshore and north of the present entrance, the attrition of the shore and offshore areas immediately south thereof, and the continuing growth of the Clatsop Spit shoals are considered, it is not difficult to perceive that this threat is quite imminent, particularly when the destructive effects of storms experienced at the Columbia Entrance are directed toward such an eventuality.

Recognizing the possibility that such a relocated entrance, if developed in accordance with careful planning, could possess merit over the present entrance, insofar as channel maintenance is concerned, the model offers an opportunity for careful and complete engineering analyses of the problems associated with the development of such a new channel. In the model the factors now producing the large shoals in the present entrance can be evaluated as they would affect conditions along the relocated channel and permit engineers to investigate all phases of the potential problem before it occurs, thus saving much time, money, and effort that might otherwise be expended. If the dislocation to establish navigation that would occur by a sudden relocation of the entrance channel is considered, one of the initial model tests will be to explore the consequences of such relocation and to determine the courses of action to be taken.

Jetty rehabilitation

As previously mentioned, jetties at the Columbia Entrance have been rehabilitated on several occasions to restore parts of these structures that have been damaged or destroyed by destructive storms. Rehabilitation undertaken in the past has largely been directed toward restoration and increasing the height and sections of those parts of the structures lying above mean sea level on the assumption that the part below this level, if not in satisfactory condition, would make itself evident by complete failure of the jetty structure. This assumption has proved correct, as subsidence of the structures, particularly after their initial construction, revealed the fact that a certain amount of consolidation was required to effect a stable jetty foundation. Such a foundation has been secured over the years until now rehabilitation is confined to restoration of those parts of the structures damaged by wave action. Notwithstanding the limited nature of rehabilitation, the work involved is costly and must be conducted during the relatively calm season of the year. Its cost and the construction difficulties involved, together with experience gained at other river entrances, have raised question among engineers as to the value of such limited

rehabilitation insofar as maintenance of project depths are concerned.

Although completely rehabilitated jetties undoubtedly provide some benefit to small craft against heavy seas as long as these craft remain in the lee of these structures, these particular benefits are seldom of sufficient magnitude to justify the large rehabilitation costs involved. Further, if sea conditions beyond the jetty are such as to preclude safe operation of small craft, there is little reason to expect that the prudent operator of such a vessel would have need for any protection a rehabilitated jetty might provide. Despite the deteriorated condition of the South Jetty at the entrance to Grays Harbor, Wash., just north of the Columbia Entrance, depths in the authorized entrance channel to that harbor are more than twice those for which the jetties were designed to maintain. Although early rehabilitation of that jetty is now being considered, this work is required to preserve its structural integrity against attack by currents that threaten to undermine the jetty foundation.

Aside from work of such emergent nature, there appears to be sound basis for question as to the actual value of rehabilitation for the sole purpose of preserving the upper parts of jetties. Certain areas exist for exploration of the most economical time interval for conducting such rehabilitation operations, taking into consideration the construction problems involved and the actual benefits to be expected. The Columbia model, in its initial testing phase, will provide an opportunity to demonstrate the value of recent rehabilitation work and such further work as may be proposed for the immediate future.

Entrance channel realinement

As may be noted from recent condition surveys, generally as shown on figure 9, the authorized entrance channel takes a southwesterly course through the outer bar as it proceeds seaward through the Columbia jetties. Although dredging on the outer bar on this alinement is not now significant, heavy dredging is required to maintain the adopted channel through the inner bar within the jetties. Just west of the authorized channel hydrographic surveys reveal a natural parallel channel of even greater depth, which appears to offer a possibility of decreasing dredging requirements if, through some means, advantage might be taken of the greater depths afforded, without introducing an undesirable risk to navigation.

In connection with model studies of different channel alinements at the Southwest Pass entrance to the Mississippi River, it was found that the introduction of a degree of sinuosity into the channel alinement took advantage of the prevailing river and ocean forces to obtain a more satisfactory entrance channel. Such an expedient, in combination with realinement of the entrance channel, might have definite merit at the Columbia Entrance and will be tested in the comprehensive model.

Salinity intrusion and shoaling

The model will accurately define the extent of salinity intrusion in the Columbia Estuary under the different conditions of tide and upland discharge and will show the part such intrusion plays in the development of the shoaling regime of the estuary and entrance. The movable-bed tests will show the mechanism responsible for the formation of Clatsop Spit, the forces involved in the great imbalance of offshore materials to the north and south of the present entrance, and the role that alongshore littoral forces play in the formation of these large shoal and attrition areas. Once these forces are clearly understood and their interrelationships evaluated in the model, tests can be made of structural measures designed to take advantage of these forces in the development of an entrance channel that will be easier and less costly to maintain.

Reduction of entrance width

Although early engineers were guided by observations of existing bank-to-bank widths in the estuary in determining the present width between the ends of the jetties, approximately 2 miles, it has long been felt by the present-day engineers that these structures were too far apart to guide efficiently the forces at work to produce an easily maintained entrance channel. In the model it will be possible to test the value of more confining structures, such as would result from extension of the alinement of the original South Jetty past the "knuckle." Other confining works, such as groins constructed perpendicular to the South Jetty, could also be tested and evaluated.

Jetty B

One of the early tests in the model will evaluate the merit of jetty B as presently authorized by the 48-foot entrance project. The 1959 prototype measurement program revealed the existence of generally upstream currents at all times along the bottom of the "cul-de-sac" in which jetty B would be located. Such a structure could possibly tend to cause undermining of the existing North Jetty through reflection of the prevailing bottom upstream currents toward that structure. The merits as well as the possible adverse effects of jetty B will be explored fully in the model.

Other tests

The above brief discussion outlines some of the early tests proposed for the comprehensive

Columbia Estuary and Entrance model. Doubtlessly, numerous other tests will be made as new problems arise through increased growth and development of the Columbia Basin. Not the least of these will be that caused by increased pollution of Columbia River waters through domestic and industrial waste disposal. It may be expected that the Columbia model, like models of estuaries elsewhere, particularly along the heavily developed eastern seaboard, will define the flushing pattern peculiar to the Columbia Estuary and will point the way to solution of this problem as well as other problems not now envisioned by engineers of this day.

Summary of Present Knowledge

The vastness of the lower Columbia Estuary and Entrance area is only one characteristic that belies the complexity of the problem of achieving and maintaining a reasonable degree of permanence insofar as navigation is concerned. The problem is further complicated by the dynamic forces of ocean salinity, littoral drift, river flow, and storm waves that are primarily responsible for the geographical formation of the area and the interactions taking place therein. These characteristics, together with the extremely large and variable range of diurnal tidal action prevailing along the Oregon and Washington coasts, give to the Columbia Estuary and Entrance a uniqueness beyond compare. The tremendous amounts of energy expended by these outsized forces in the entrance and estuary area exceed those expended in any similarly improved area anywhere in the world today. Past efforts to regulate these forces in the interest of navigation have led to construction of massive and long, stone training structures, the North and South Jetties, which confine the ship channel to a relatively limited area but do not assure adequate depths for navigation without continual dredging year after year.

Comparison of depths shown by hydrographic surveys undertaken since the beginning of improvement reveal that the estuary is gradually becoming shallower, being filled with bottom materials carried by the river and transported from adjacent ocean areas. Clatsop Spit shoals, which continually encroach on the present entrance channel, constitute the most striking evidence of this progress. Other evidence, although less spectacular, such as the changes occurring immediately upstream from these shoals, confirm this trend. Measurements of current direction and velocity, as well as salinity and temperature, at different depth levels throughout the estuary reveal the existence of a salinity block or salt-water wedge action along the bottom through the entrance, which not only prevents the movement of bottom materials to the sea but encourages the movement of ocean materials into the estuary. This intrusion of salinity into the estuary, an incidental byproduct of improvement, now controls the regimen of the estuary area.

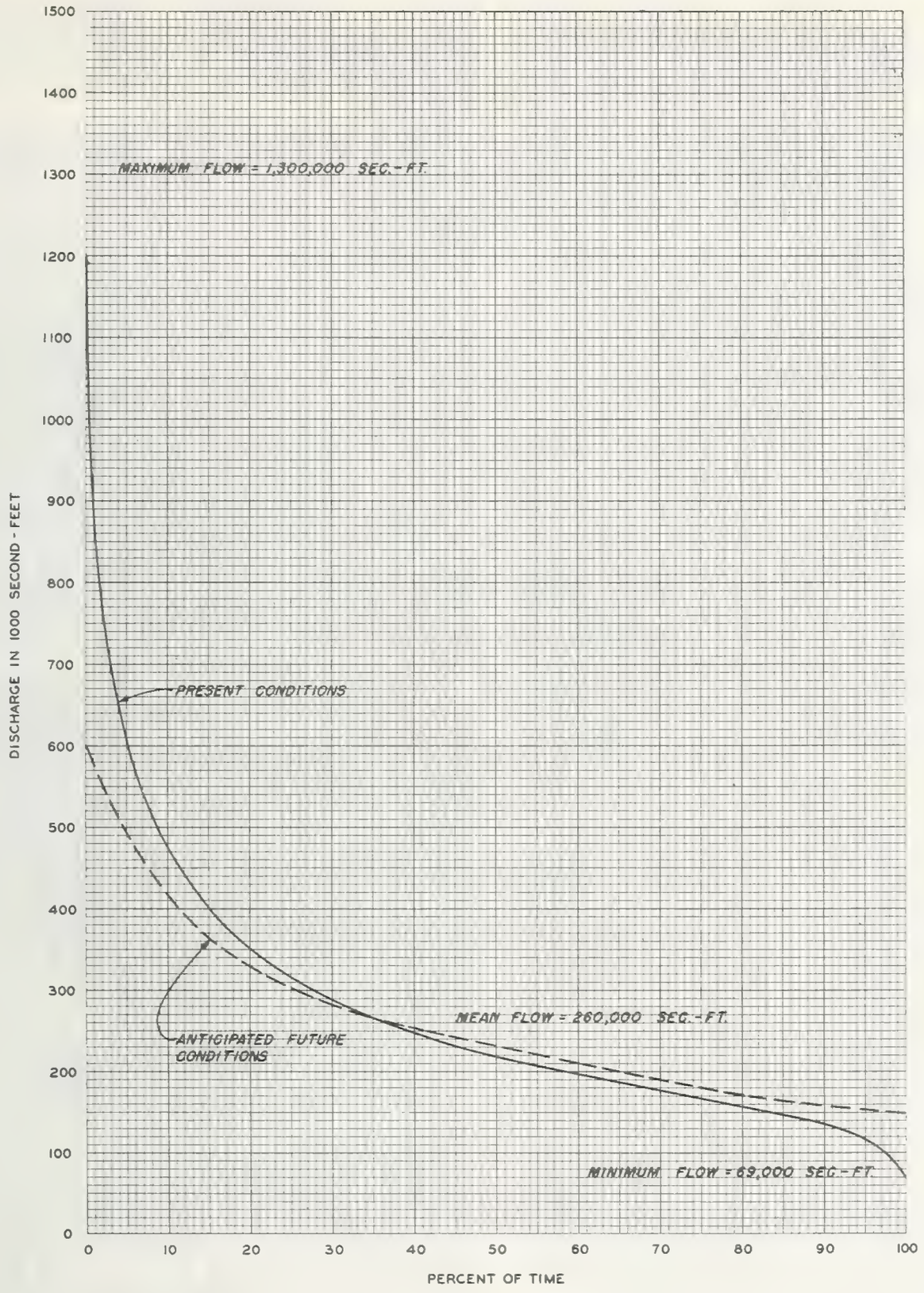
Another incidental effect of improvement, interception of the predominantly north to south alongshore littoral drift by the North Jetty, has created a severe imbalance of materials immediately offshore the entrance. This imbalance threatens not only to pinch off the present entrance channel by the accumulation of littoral material from the north but, with the assistance of storm waves, also threatens to produce a major relocation of the river outlet to the south, where attrition of the shore and offshore area has weakened the confining influence of the South Jetty.

The comprehensive hydraulic model of the Columbia Estuary and Entrance, now undergoing verification tests at the Waterways Experiment Station, offers the sole means of developing an engineering analysis and evaluation of the forces responsible for the pending cataclysm in this area and of determining the action which might be taken to prevent such an eventuality.

Considerations for the Future

Achievement of all feasible storage in the Columbia River headwater areas in the primary interests of flood control, hydroelectric power, and irrigation will alter materially the pattern of river flows in the lower estuary. As shown on figure 28, these flows have in the past ranged from a minimum of 59,000 c.f.s. to a maximum of about 1,300,000 c.f.s. Regulation by such storage will reduce the future range of these flows by lowering extreme discharges to not more than 600,000 c.f.s. and by increasing minimum flows to about 150,000 c.f.s. This alteration of the river flow pattern may be expected to eliminate any possible flushing of bottom sediments from the estuary to the sea by extreme river discharges of the magnitude that has been experienced in the past.

On the other hand, the increase in low river flows may, to a small degree, reduce the extent of salinity intrusion in limited parts of the lower estuary. While this regulation of river flow is not expected to modify appreciably the total amount of fresh water entering the estuary, it will, by reduction of extremely high discharges, tend to maintain the permanency and quite likely increase the degree of salinity intrusion into the estuary. Also, as revealed by experience gained at the Savannah Estuary, such regulation of upland flows may tend to concentrate shoaling to a much more limited length of the Columbia Estuary.



Notes:

1. *This flow-duration curve is for Columbia River at mouth.*
2. *Maximum, minimum and mean values are computed from observed data.*
3. *Drainage area is approximately 259,000 sq.mi.*

FIGURE 28. — Flow-duration curve.

History of the Pacific Northwest reveals that channels of greater and greater depth are required to support the increase in ocean commerce attending the continued growth and economic development of this region. Due to the geography of the entrance area and the severe wave and weather conditions that prevail in that area during the winter months, depths of 48 feet are necessary for safe passage through the Columbia Entrance, while the main Columbia channel inland now provides only a depth of 35 feet. Recent increases in the size and draft of the world fleet of ocean carriers have revealed a need for increasing the minimum depth of the main river channel above the entrance to Portland and Vancouver from 35 to 40 feet, and plans to effect such further deepening of the main channel are now underway.

Although no corresponding deepening of the entrance channel is planned at present, due to the fact that elimination of delays caused by the 48-foot entrance channel to prospective traffic will not now justify such further deepening, it is obvious that eventually, as the region continues to grow, greater entrance depths will be required. Provision of the contemplated 40-foot channel in the river above the entrance is expected to extend the effect of salinity intrusion farther into the estuary, causing changes in shoaling patterns and increasing dredging to maintain the larger channel dimensions. Any further deepening of the present entrance channel will intensify saline conditions within the estuary and certainly modify, and quite likely aggravate, the shoaling problem.

As previously mentioned, pollution of the estuarine waters is not now a serious problem facing those charged with maintaining the quality of Columbia River water. However, with the expected growth in population and industrial development in the area tributary to this great river, experience elsewhere, particularly along the Atlantic seaboard, points to the eventual need for strict control of pollutants disposed in the Columbia River. The extent of such control will largely be governed by the flushing characteristics of the Columbia Estuary as affected by improvements designed primarily for the sole benefit of navigation. It does not appear unreasonable that the degree of pollution induced into the river may some day exceed the flushing capacity of the estuary.

All these problems are susceptible to study, analysis, and evaluation in the comprehensive hydraulic model of the lower Columbia Estuary and Entrance. There is little doubt that the model will contribute greatly to the solution of these and other problems of an unforeseeable nature at this time. In the event the model demonstrates the merit of a planned major relocation of the entrance channel through the South Jetty, it appears probable that many of the problems created by salinity intrusion into the estuary may also be solved or greatly alleviated.

Such a planned relocation would involve construction of a navigation canal in the Point Adams area of the dimensions required by ocean commerce connecting the estuary at that point with the deep ocean area just south of the present South Jetty. A lock might be required in this canal at Point Adams to eliminate or reduce the intrusion of salinity into the estuary from the canal. Also, a new but substantially shorter South Jetty may be required to fix firmly the entrance channel between this new structure and the enrockment forming the present South Jetty. With such a relocated entrance channel and salt-water lock, the present entrance would be abandoned, closed to navigation, and allowed to shoal naturally. If shoaling in the present entrance channel reduces controlling depths to the equivalent of those prevailing prior to 1885, it may be expected that the intrusion of salinity into the estuary will be correspondingly reduced until it is no longer a controlling factor in the estuarine regime.

With the reduction of depths created by shoaling it may be expected that, even with the minimum regulated flow of about 150,000 c.f.s., river flows will completely usurp the entire channel section at all times and there will no longer prevail the marked density currents that are now so characteristic of the intrusion of salinity into the estuary. At that time, except for the modification of the river flow pattern resulting from upstream storage, the restored flushing characteristics of the river would move the bottom sediments to the sea in a manner similar to that which prevailed prior to man's attempt to improve the entrance. The freshwater estuary thus obtained will be capable of more complete flushing of pollutants, and it is reasonable to expect that dredging requirements for navigation will be considerably less demanding.

Along with such achievement there would be restored the uninterrupted predominantly north to south littoral drift past the present entrance that would nourish and restore the starved shoreline immediately to the south. Although restoration of the natural littoral regime may eventually require dredging of the relocated entrance channel, this work will not be required for many years due to the depths now existing in that area. Thus, with restoration of conditions which largely prevailed prior to improvement, the strategic value of the Columbia Estuary and Entrance area may be assured throughout the foreseeable future at reasonable costs.

Acknowledgment

The discussion and data presented herein were obtained from research conducted under the Engineer Planning Program of the U.S. Army Corps of Engineers. The permission granted by the Chief of Engineers to publish this information is appreciated.

MEANS AND METHODS OF INDUCING SEDIMENT DEPOSITION AND REMOVAL

[Paper No. 69]

By JOHN W. HARRIS, *supervisory hydraulic engineer, U.S. Army Engineer District, Savannah, Ga.*

Synopsis

Maintenance of Savannah Harbor has become progressively more critical as the deepening of the channels caused heavy shoaling concentrations in a highly industrialized section of the harbor where spoil disposal areas are limited. The Savannah Harbor model study developed means of inducing sediment deposition in areas where more economical means of removal could be utilized. The hydraulic changes expected to be produced by the proposed plan of improvement and their effects on shoaling, as well as their effect on the efficiency and cost of harbor maintenance, was discussed.

Introduction

Savannah Harbor, Ga., is located in the lower 21 miles of the Savannah River. Maintenance of Savannah Harbor has long been a problem of concern to navigation interests and an item of great expense to the Government. Concurrent with the improvement of the harbor, each successive deepening has caused the shoaling to shift upstream. After the harbor was deepened to 34 ft. in 1947-48, shoaling concentrated in channels adjacent to the city of Savannah, where only limited areas are available that can be used for spoil disposal. To provide adequate maintenance of the channel without excessive increases in cost, it was evident that action to reduce or relocate the shoaling would be required to insure adequate maintenance of the channels at reasonable cost. The Savannah Harbor investigation and model study was performed for this purpose. The model studies have developed a plan to reduce shoaling in the navigation channels and to induce the deposit of most of the shoal material in a sediment trap that would be located adjacent to adequate spoil disposal areas.

Description

Savannah Harbor (fig. 1) is located in the tidal estuary where the Savannah River empties into the Atlantic Ocean. Channel locations refer to 1,000-ft. stations with the upper limit of the harbor at station 81+499 and the lower limit at station 245+000, which is generally referred to as station 245. Similar reference is made to other 1,000-ft. stations throughout the harbor. The authorized maintenance project provides for a channel 36 ft. deep at mean low water and 500 ft. wide from deep water in the ocean across the bar to station 194 (harbor entrance) ; thence 34 ft. deep and generally 400 ft. wide to station 94 (Georgia Ports Authority terminals) ; thence, 30 ft. deep and 200 ft. wide to the upper limit (Atlantic Coastal Highway bridge). Turning basins are located in the vicinity of stations 83, 90, 95, and 104. In the vicinity of the Atlantic Coast Line Railroad terminals, the channel is 550 ft. wide. The navigation channels in the inner harbor are located in North Channel and Front River. The Atlantic Intracoastal Waterway crosses North Channel at about station 165. The center of the city of Savannah is located on the south side of Front River at about station 120 with industrial developments extending to the upper limits of the harbor.

Shoaling

The annual shoaling rate for the inner harbor averages about 7,000,000 cu. yd. The greatest concentration occurs in Front River opposite the city of Savannah, as shown on figure 1, where over 50 percent of the shoaling takes place between station 117 and station 136. The shoaling shifted upstream to its present location after the deepening of the channels to 34 ft. in 1947-48. Figure 2 shows a graphical comparison of the shoaling before and after the deepening. The shoal material is primarily silt and clay with very little sand-size material.

Maintenance

Pipeline dredges are used to maintain the inner harbor channels. A Government dredge is normally assigned to the district, although frequently it is necessary to obtain the services of a privately owned dredge under contract to assist the Government dredge. The present maintenance program for the inner harbor (station 194-81+499) costs over a million dollars annually and requires an average of 11 months work annually by Government plant and 2 months work each second year by contract. Dredging requirements are based on the present

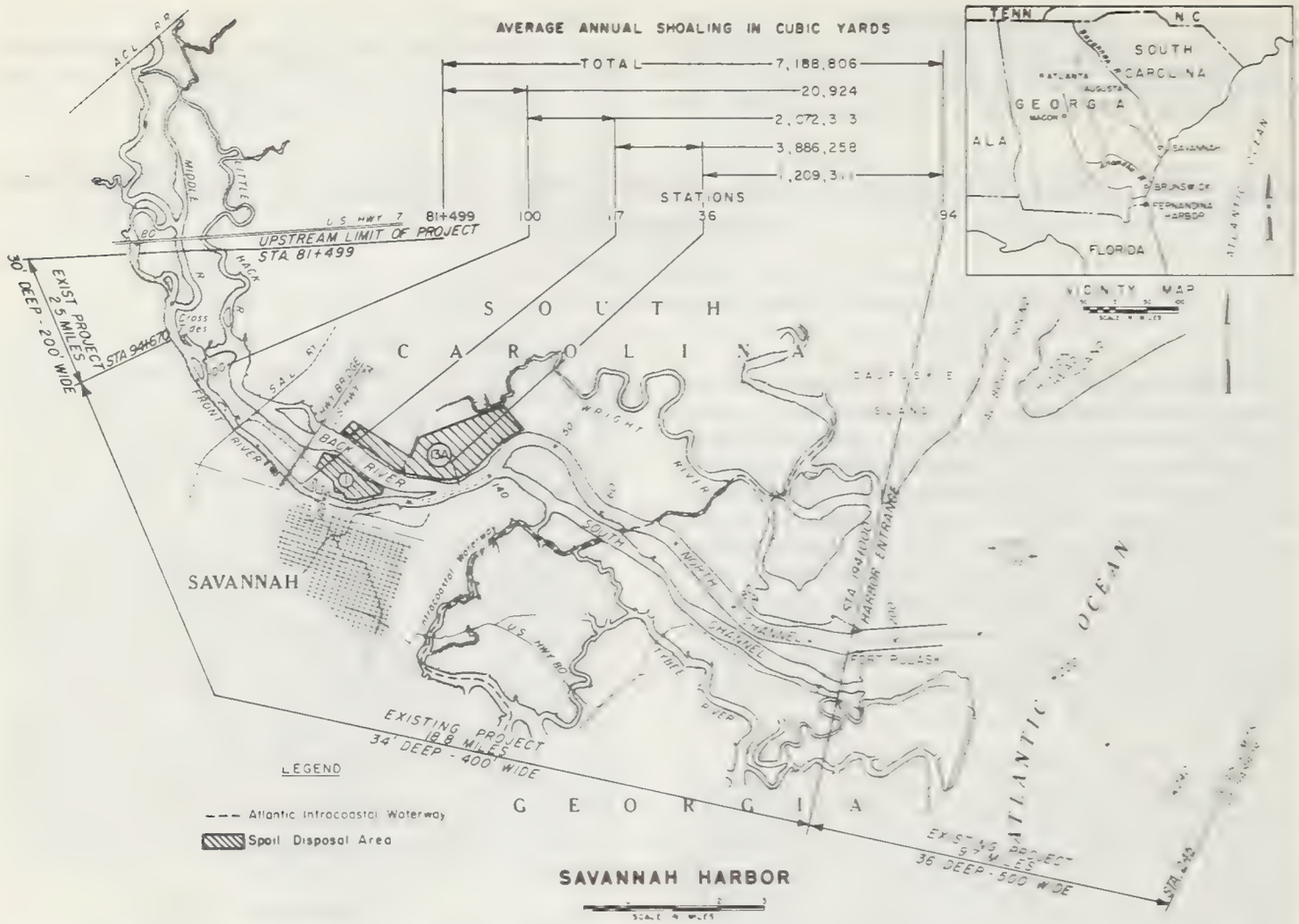


FIGURE 1. — Existing project and annual shoaling.

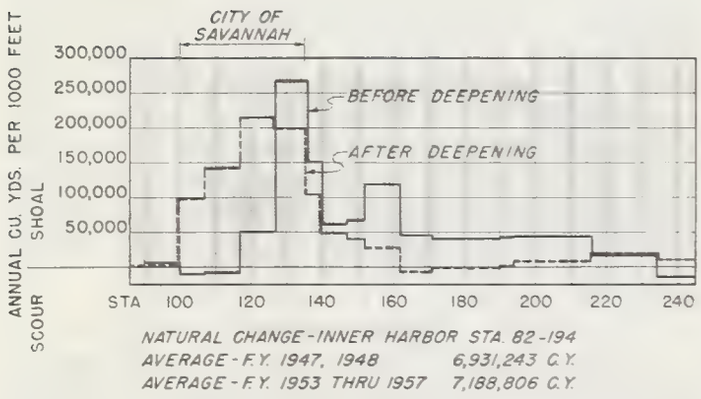


FIGURE 2. — Annual shoal and scour (natural change) before and after deepening navigation channels.

maintenance policy, which provides that dredging be performed when an area with authorized project depth of 34 ft. has shoaled to about 30 to 32 ft. Maintaining full project dimensions would require more frequent dredging than the present policy. Maintenance of the bar and jetty channels is performed with Government-owned hopper dredge. The annual shoaling on the bar (station 194-245) amounts to about 700,000 cu. yd.

Spoil Disposal Areas

The shortage of spoil disposal areas adjacent to the rapidly shoaling channels has greatly increased the cost of channel maintenance. Spoil area No. 1 (fig. 1) has been virtually filled and area 13A has been diked and is now being used for maintenance of the major shoal sections of the channel, although pipelines in excess of 10,000 ft. are required for maintenance of some channel sections.

Description of the Savannah Harbor Model

Approximately 413 square miles of prototype area were reproduced in the Savannah Harbor model, including the Atlantic coast from Hilton Head Island to Wassaw Sound and offshore areas well beyond the 40-ft. contour of the ocean; up the Savannah River about 50 miles to the head of tidewater at Ebenezer Landing; the extensive system of tidal tributaries, salt-water

creeks, and boundary marshes that affect tidal action throughout the harbor; and the Atlantic Intracoastal Waterway from Calibogue Sound through Skidaway River. The model was constructed on scale ratios, model to prototype, of 1:800 horizontally and 1:80 vertically. It was equipped with necessary appurtenances to reproduce and measure all pertinent phenomena such as tidal elevations, salt-water intrusion, current velocities, fresh-water inflow, pollution, and shoaling. The model was constructed to conform to the prototype conditions that existed in 1950. Hydraulic and salinity verification was based on extensive protoype measurements made in 1950 and 1951. Shoaling verification was based on the quantitative distribution for the years 1953-54, which is representative of prevailing conditions for the existing 34-ft. project depth.

Shoaling Tests

The procedure developed and used in the general investigation for duplicating prototype shoaling involved the injection of a measured mixture of ground gilsonite and water into the model and pooling the water after a prescribed number of tidal cycles. The gilsonite that settled within the channel limits was retrieved and measured and the results expressed by a percentage distribution curve. An acceptable reproduction of prototype shoaling was attained, since exact duplication of the prototype shoaling was neither possible nor necessary. At the termination of a shoaling test all material in motion was deposited immediately in place. Dredging and spoiling techniques that are believed to have a definite effect on the prototype shoaling quantity and rate could not be reproduced in the model. However, it was not necessary to obtain exact reproduction of prototype shoaling, since the basic model shoaling distribution curve was used for evaluating the effectiveness of proposed improvement plans.

Back River Sediment Basins

The results of the verification and preliminary tests confirmed that sediment basins might offer a possible means for reducing maintenance costs if the location, entrance, and hydraulic conditions of such basins could be designed to attain the maximum possible rate of shoaling in them and if the basins could be located adjacent to adequate spoil disposal areas. Back River is ideally located for construction of a sediment basin. It has no commercial traffic, it is located near the major shoal area, it has no industrial development on either side, and the Government has perpetual spoilage easements on extensive diked areas adjacent to the proposed location for the sediment basin. A series of tests were made to determine the most favorable conditions for a sediment basin and the effect of such a plan on the hydraulic regimen of the harbor.

Development of a Plan

The effectiveness of each plan was judged primarily by the ability of the plan to reduce shoaling in the Front River navigation channel between stations 82 and 135 and to concentrate shoaling in the sediment basin. The model tests of a sediment basin, located in the lower part of Back River, 1,500 ft. wide and 2 miles long with an entrance channel in the mouth of Back River, reduced the shoaling in the navigation channel by about 20 percent. Since the reduction in channel shoaling occurred downstream from the entrance to Back River instead of in the critical shoaling area upstream from that point, little benefit would be realized.

Two tests were made with the entrance channel to the sediment basin located in the immediate vicinity of the shoaling concentration and with constriction dikes in the mouth of Back River, which were necessary to make the plans effective. One test was made with the entrance channel located near the middle of the major shoal area at station 125 and with a 150-ft. opening between the constriction dikes as shown on figure 3. The sediment basin retained 35 percent of the shoaling, and the navigation channel shoaling was reduced to 64 percent, but the reduction occurred downstream from the entrance to the basin with a slight increase in the upstream quantity. The other test was made with the entrance channel located near the upstream end of the major shoal area at Station 116 and with a 200-ft. opening between the constriction dikes, as shown on figure 4. The shoaling in the navigation channel was reduced to 65 percent while the sediment basin retained 38 percent. Although these plans were successful

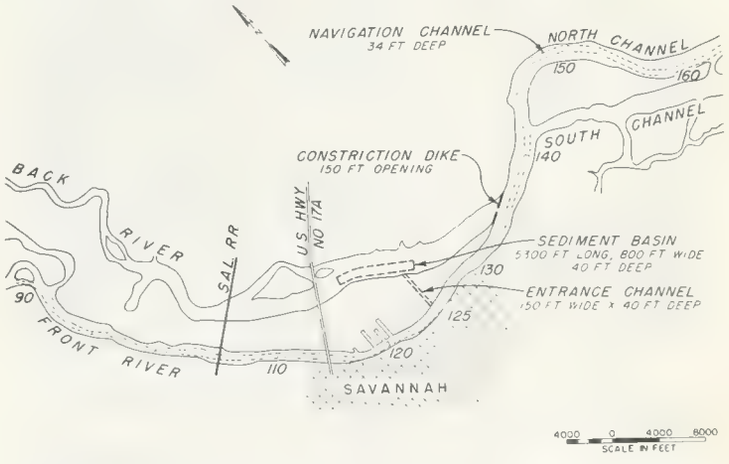


FIGURE 3. — Sediment basin with entrance channel at station 125.

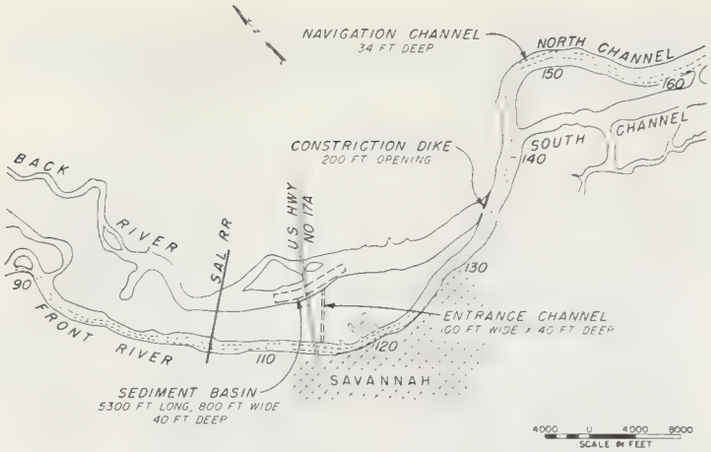


FIGURE 4. — Sediment basin with entrance channel at station 116.

to some degree, substantial shoaling would still occur in the area where spoil areas are critical.

Tests were made with a 300-ft. wide entrance channel in the mouth of Back River and a tide gate in Back River above the SAL Bridge about 1.5 miles upstream from the sediment basin, with a canal connecting the Back River tidal prism, above the tide gate, with Front River. The tide gate served to reduce the ebb flow through the sediment basin and also to increase ebb flows in Front River to prevent shoaling. This plan (fig. 5) was very effective, but the

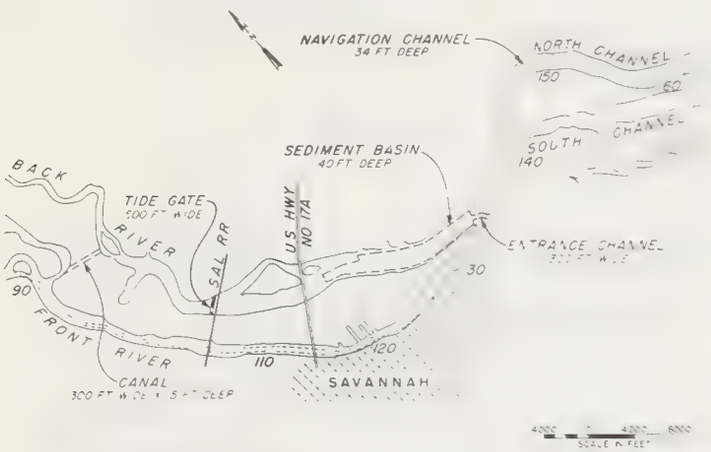


FIGURE 5. — Sediment basin with tide gate in upstream location.

sediment basin was enormous and the initial construction cost too high to produce a favorable economic ratio. It was determined by additional tests that the sediment basin could be reduced to a width of 600 ft. and still be reasonably effective. Up to this point the tests were made for the present authorized channel depth of 34 ft.

To determine the effectiveness of the sediment trap-tide gate plan with anticipated future harbor improvements, tests were run that incorporated deepening of the channels to 40 ft. These tests indicated that the ebb velocities in Front River were inadequate to prevent shoaling. The tide gate was then shifted 1.3 miles downstream to a point at the upstream end of the sediment basin to increase the volume of the Back River tidal prism, which discharged into upper Front River. This arrangement was satisfactory for the 40-ft. deep channel; however, when it was tested with the 34-ft. depth in the navigation channel, spring tide velocities in Front River were observed that would be hazardous for navigation. This condition was corrected by placing a side opening in the tide gate, which reduced the effective volume of the Back River tidal prism. These tests showed conclusively that the sediment trap-tide gate plan could be used for present project channels and also adapted to future harbor developments.

Side Opening in the Tide Gate

The side opening in the tide gate structure, which is required to prevent excessive velocities in the existing navigation channel, demonstrates the hydraulic mechanics that reduce the shoaling in Front River. An opening of 500 ft. was successful in maintaining the ebb velocities compatible with the requirements of navigation, but the plan was not effective in reducing the channel shoaling nor for inducing shoaling in the sediment basin. An opening of 200 ft., however, was effective in reducing shoaling in upper Front River; but the sediment basin was relatively ineffective. An opening of 100 ft. was effective in reducing the shoaling in Front River, making the sediment basin effective and preventing excessive velocities in Front River.

The Sediment Trap-Tide Gate Plan

The most effective plan was phase 8, scheme 6, test 4 (fig. 6), which requires a sediment basin in Back River 600 ft. wide, 40 ft. deep,

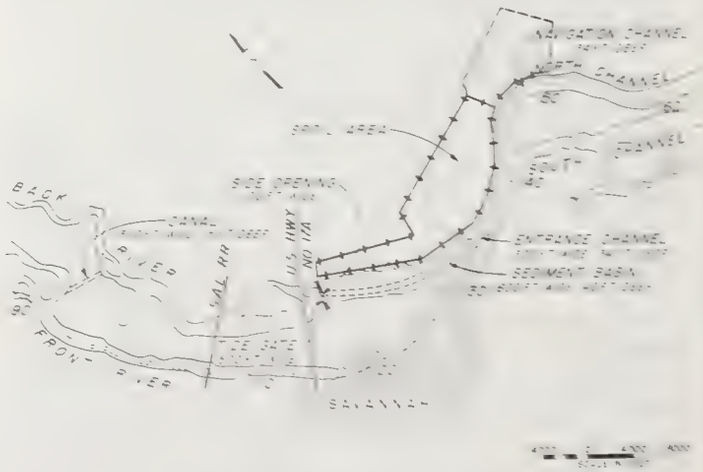


FIGURE 6. — Sediment basin with tide gate in downstream location (phase 8, scheme 6, test 4, and test 10).

and about 2 miles long; a tide gate in the vicinity of Highway 17A; and a canal connecting Back River to Front River in the upper part of the harbor. The entrance channel to the sediment basin is 300 ft. wide and 34 ft. deep. During the shoaling tests, at the end of the prescribed number of tidal cycles, a stable condition had not been reached in the navigation channels and sediment was still actively entering the sediment basin, whereas, in other tests the sediment had stabilized. Test 10 was then performed which extended the shoaling test until stability was reached. This plan reduced the shoaling between stations 82 and 135 from 77 to 5 percent and between station 135 and station 205 from 23 to 9 percent.

The Tide Gate Structure

The tide gate structure was provided to increase the efficiency of the sediment basin by reducing ebb flows through the basin. A 500-ft. gated opening was provided in the structure that opened automatically to allow flood flows to pass through but which was closed during the ebb periods. A canal was provided to drain the Back River tidal prism above the tide gate structure into Front River, thus increasing ebb flows in Front River. During spring tide periods it would be necessary to pass a part of the water from the Back River tidal prism through the sediment basin to prevent high velocities in Front River that might be dangerous to navigation. The shoaling tests were conducted with a fixed side opening 100 ft. wide in the tide gate structure; however, the efficiency of the sediment basin probably could be increased by providing an opening only during the spring tides that produce excessively high velocities.

Sediment Basin Maintenance

Based on the present harbor shoaling rate and the model shoaling tests, it will be necessary to dredge about 6 million cu. yd. annually from the sediment basin. To retain the effectiveness of the sediment basin, it will be necessary to dredge it annually or probably continuously, with periodic maintenance dredging in the navigation channels. The redistribution of the shoaling material is expected to produce considerable savings over the present maintenance cost because:

(1) If a conventional pipeline dredge is used, reasonably short pipelines would be required, since the spoil area is located adjacent to the basin, whereas long lines are now required to maintain navigation channels.

(2) Fixed pipelines could be utilized to eliminate much labor in handling.

(3) The dredge would be out of the navigation channel, which would eliminate loss of time permitting vessels to pass.

(4) The operation of the dredge would be confined to limits of the sediment basin, eliminating the costly movement of the dredge over many miles of navigation channel.

(5) It would be possible to let the sediment consolidate to an optimum density for efficient removal from the basin; whereas, the present specified depths in the navigation channel, which must be maintained, often necessitates dredging material that has not consolidated sufficiently for economical removal.

The savings in labor and increased efficiency of dredging the material from the sediment basin would produce appreciable reduction in the maintenance cost. Preliminary economic studies for the plan indicate a favorable benefit-cost ratio. Although no specific plans have been made, it is contemplated that some means other than a conventional pipeline dredge can be devised to maintain the sediment basin, such as a simple arrangement of pumps on barges with drags similar to the hopper dredge or dustpan-type intake, or semimobile dredge powered by electricity. Since the preliminary economic studies have been based on the maintenance of the sediment basin with a conventional pipeline dredge, it is almost certain that final studies will indicate a more favorable benefit-cost ratio for the plan.

Improved Channel Maintenance

The present maintenance of the navigation channels requires the services of a 24-inch pipeline dredge almost continuously to dredge the 7 million cu. yd. annually at a cost of about $1.25 million. The sediment trap-tide gate plan would reduce the channel shoaling to the extent that dredging would be required only periodically and channel conditions would be greatly improved. The project channel depths, which would be almost self-maintained, is a very important benefit.

Verification of Model Shoaling Test With Prototype Data

The shoaling tests for the sediment trap-tide gate plan are partially verified by prototype data, which indicate that Savannah River discharges of 16,000 c.f.s. or greater tend to shift the concentration of shoaling downstream and that discharges of this magnitude, if continued for an appreciable length of time, have a pronounced effect on shoaling in the harbor.

Figure 7 shows Savannah River discharges for about a 1-year period with center-line profiles in the harbor during various flow conditions. Fluff (unconsolidated sediment) is present in areas susceptible to shoaling. During a period of normal discharges up to 10,000 c.f.s. the fluff extended from station 103 to station 140. Moderately high discharges shifted the

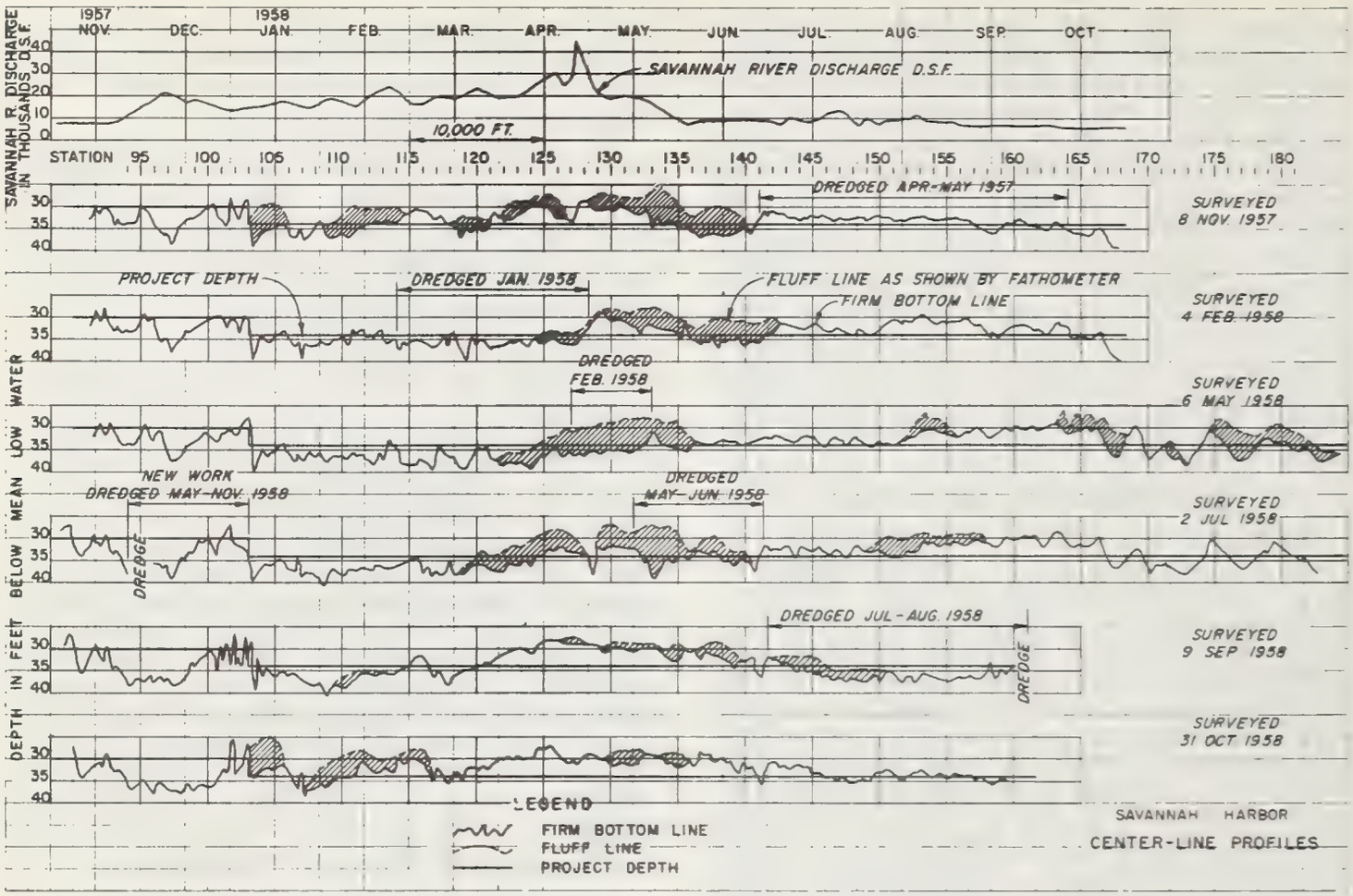


FIGURE 7. — Effect of Savannah River discharges on shoaling.

fluff downstream, and after several months of high flows the fluff extended from station 122 to station 184. Then when the discharges returned to normal the fluff moved back upstream with rapid consolidation of material occurring between station 120 and station 140. The sediment trap-tide gate plan produced hydraulic conditions in the model that eliminated shoaling in Front River similar to the effect produced by high discharges.

Conclusions

Shoaling, which the deepening of the Savannah Harbor shifted to a section where spoil disposal areas are limited, prompted model studies which have demonstrated means of inducing shoaling in a sediment basin which is located adjacent to adequate spoil areas.

The most advantageous plan developed provides for a sediment basin and tide gate in Back River. In the model this plan was successful in increasing ebb flows in Front River, which shifted the shoaling from this critical area to the sediment basin. The model tests indicated that 89 percent of the shoaling would be induced in the sediment basin.

Considerable savings in harbor maintenance would be realized through the efficient dredging methods that would be possible in maintaining the sediment basin.

SEDIMENTATION IN THE SAN FRANCISCO BAY SYSTEM

[Paper No. 70]

By BERNARD J. SMITH, *civil engineer, U.S. Army Engineer District, San Francisco*

Abstract

The San Francisco Bay system is a landlocked series of embayments of 430 square miles. The annual sediment load approximates 8 million cubic yards, 85.5 percent from the Sacramento-San Joaquin River system and 14.5 percent from Bay area streams. Upon entry into the Bay system, the character of the sediment inflows change. The density of the bedload decreases from 90 to 31 lb. per cu. ft. for San Pablo Bay, 49 lb. per cu. ft. for Central Bay, and 88 lb. per cu. ft. for Suisun Bay; and the sand content is lost and the silt content increased an additional 14 percent. The clay content remains constant at about 57 percent. For fresh water inflows of around 30,000 c.f.s., the dilution factor on entering the system is about 1:54. However, no discernible drop in parts per million of sediment load has been observed. It is concluded that tidal- and wind-wave-induced turbulence erodes bottom deposits and inhibits settlement. Contribution of the supplemental supply from littoral erosion outside the Golden Gate is discounted. Historical sedimentation evidences cyclical trends and interactioning within the system. The effects of wind action on marginal and intermediate shoals are noted. Massive displacements of sediment volumes would drastically affect the biota of the estuary.

General

The sedimentation pattern, a critical economic factor for an estuary of commercial rank, becomes of added importance in a system such as the San Francisco Bay complex in which environmental change may have far-reaching consequences on the biota of the region. The Bay system is the gateway in the spawning season for salmon runs to the headwaters of the Sacramento River system. It also is a strategic landing area for wildfowl on the Pacific flyway. These aspects are important to the regional economy and of increasing import to the State and Nation.

The complex of delta, bays, and straits making up the San Francisco Bay system is the tidal terminus of an estimated annual mean natural runoff of 34,881,000 acre-feet (*1*). This flow approximates 49 percent of the estimated total natural mean annual runoff of the State of California. Concomitantly, the system is the natural receiving basin for large sediment inflows. However, changes have been extensive, in the last 100 years particularly, so that the full natural runoff is not discharged into the system, much being diverted, and the sedimentation pattern consequently has changed. In addition, cyclic phenomena, such as droughts and floods, underlie some fluctuations in sediment inflow, deposition, and scour.

[1] DAINES, N. H. STUDY OF SUSPENDED SEDIMENT IN THE COLORADO RIVER. U.S. Dept. Int., Bur. Reclam. Mimeo. 26 pp. Sept. 1949.

The Source and Character of the Sediment Inflow

Figure 1 gives the soil conditions over the contributory watersheds and the historical outflow into the delta, the component bays, and the discharge through the Golden Gate for the period 1924–60, taking into account the operation of the reservoirs constructed in the period. Table 1 gives the percentage breakdown by five classes of soil cover for the tributary sediment source area.

TABLE 1.—*Soil classification of sediment source area tributary to Suisun Bay, San Francisco Bay System*[1]

Soil type	Area	Percent of total
	Square miles	
Sandy loams	23,305	39.5
Loams	6,490	11.0
Clay loams	21,004	35.6
Clays	3,127	5.3
Variable	5,074	8.6
Total	59,000	100.0

[1] Data obtained from reference (*2*).

The effect of diversions for municipal, industrial, and agricultural use, together with transfer of flows south through the Delta, is apparent in the 37-year average of historical outflows to the Pacific — an outflow through the Golden Gate of 21 million acre-feet as against a full natural estimated average of 35 million acre-feet. Although sediment inflow would not decline in strict proportion to declining outflow, nevertheless a substantial drop would occur.

Reservoir retention and regulation of flow, however, does not necessarily imply equivalent reduction in sediment transport to the lower bays. Floodflow releases through dams scour a portion of the trapped sediment. In the dry season the clear water releases tend to regain full competency to carry the maximum sediment load by erosion below the dams. Daines,[1] in studying sedimentation in the Colorado River system, noted that the monthly sediment loads varied considerably, with the floodflow months showing the highest loadings. Similar observa-

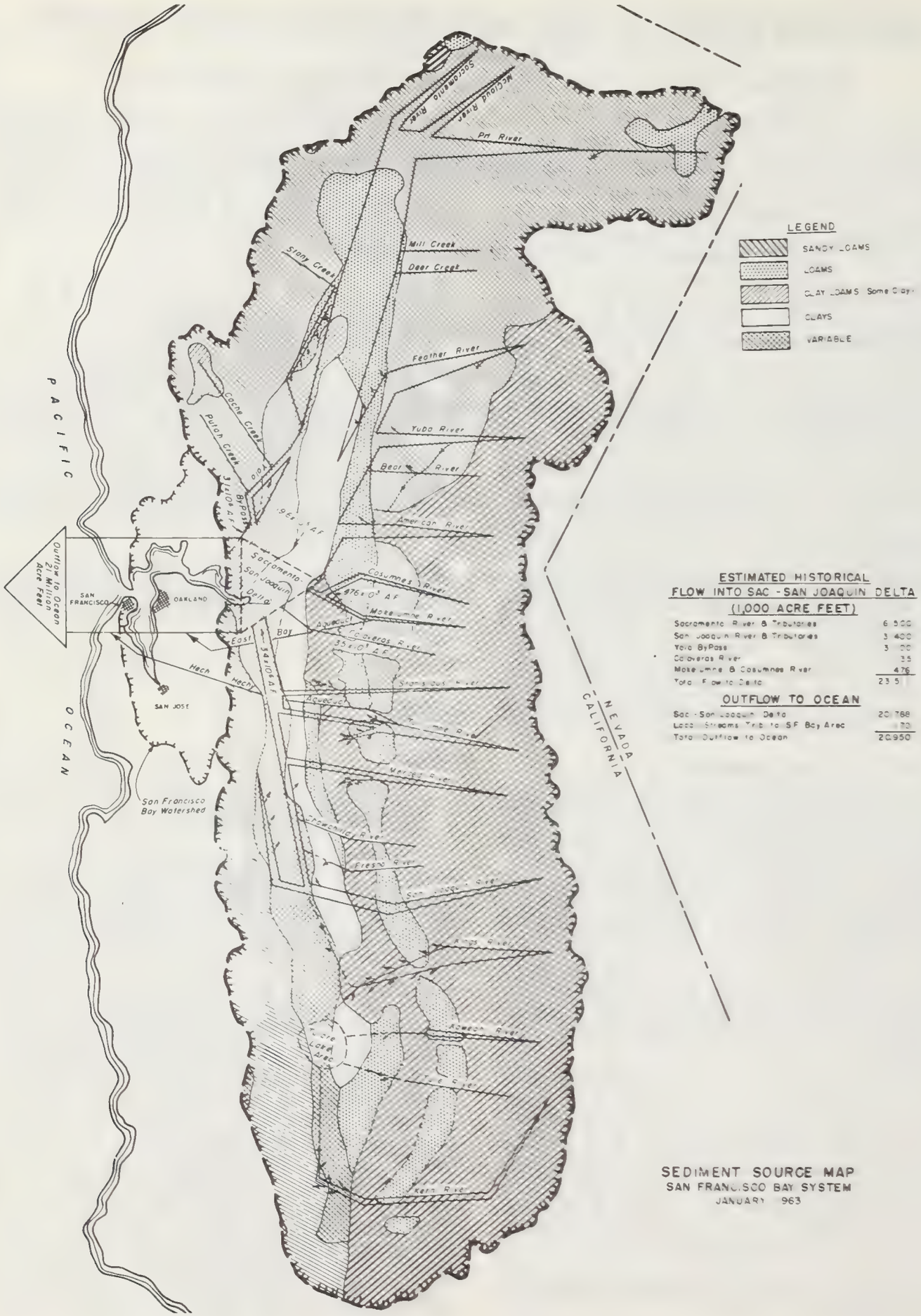


FIGURE 1. — Source of sediment of San Francisco Bay system, 1963.

tions have been recorded for Central Valley streams of California. Hence, the source of sediment can be regarded as being over the entire watershed, above and below reservoirs, with the floodflows carrying through reservoirs into downstream channels much scour material from the uplands. Figure 2 shows the major rivers and major dam sites for the Central Valley (dams constructed or authorized) and indicates the areas that normally would contribute to the sediment inflow to the Delta, i.e., areas below the lowest major dams, and the additional areas that in times of released floodflows would contribute sediment from the upland regions.

It is apparent that in years of no major flood releases, the character of the sediment inflow would reflect the soil conditions of the piedmont and alluvial fan areas in the lower elevations of Central Valley, with a high clay content and a high colloidal clay suspension (see fig. 1). In flood periods, the composition would change to reflect heavier concentrations of sand and gravel from the uplands and from scourings of channel beds. In this connection, the last upstream deposits from the hydraulic mining era should be considered. Gilbert (*7*) estimated the total debris from mining and rain erosion for the 65 years, 1850-1914, at 2,375 million cubic yards within the area tributary to Suisun Bay, distributed as shown in table 2.

TABLE 2.—*Distribution in 1914 of debris from mining and erosion from lands draining to the San Francisco bay system for the period 1850–1914*

[Estimated by G. K. Gilbert (*7*)]

Location	Debris
	Million cubic yards
Deposits—	
Within the Sierra Nevada	265
In the piedmont	520
In the channels of valley rivers	100
On inundated lands, including tidal marshes	294
In the bays	1,146
In the ocean	50
Total	2,375

The last three items of table 2, totaling 1,490 million cubic yards, represent mining debris and fluvial sediment that had been transported past the Delta in the bay system proper between 1850 and 1914, or a rate of movement under 23 million cubic yards per annum. The competency of the rivers were assumed to average at the same rate for 1914–60, an additional 1,054 million cubic yards would have been moved past the Delta. This would be insufficient to move the residual 885 million cubic yards remaining from the 1850-1914 period and the erosion loading that logically could be expected to increase from the Gilbert estimates of approximately 11 million yards per year when agricultural development was less intensive. However, of the 885 million, 435 million are considered by Gilbert to be permanently lodged — 100 in the mountain and 335 million in the piedmont. On this basis he concludes that within 50 years (i.e., by 1964) the mining debris load will have been cleared and erosion sediment, approximately 8 million cubic yards per annum, would constitute the sediment loading into the Delta.

This estimate by Gilbert is a remarkably close forecast of actual conditions as determined by the U.S. Geological Survey over the period 1957-59. The conclusion reached by the Geological Survey was that the annual average sediment inflow from the Central Valley and local streams would be 8 million yards, based on 8.8 million yards per annum for the 1957-59 water years (*11*).

Estimated Future Sediment Inflow

From the preceding reports it can be inferred the mining debris from the hydraulic mining era no longer presents a problem and any contributions from the lodgments in the uplands will be negligible. Future sediment inflow will be from the normal sources. The U.S. Geological Survey investigation (*11*) determined the average daily total load for 1957-59 to be 17,000 tons and the estimated future total load as 16,000 tons. Inflow into the Delta was 14,200 tons per day, and the future inflow was estimated at 13,000 tons. The critical aspect of converting the measured and estimated sediment tonnage into volume was based on 50 lb. per cubic foot for the suspended load and 90 lb. per cubic foot for the bedload. With these conversion factors, the Delta inflows become 7.2 and 6.9 million cubic yards, respectively. For the entire system as previously mentioned, the volumes are 8.8 million cubic yards for the 1957–59 period and 8.0 million for the future.

The USGS determinations of specific weight, based on the average particle size of the material transported by the Sacramento River, were computed by methods given by Lane and Koelzer (*11*) and Hembree.[2] These methods consider the sediment as consolidated in reservoirs under varying types of operation, the initial deposition density increasing with time and with exposure to the air (for sediment exposed during drawdowns). For sand (between 0.05 and 1.0 mm.) no increase occurs with time or exposure. For clay (particles less than 5 microns) and silt (particles between 5 microns (0.005 mm.) and 0.05 mm.), the initial density

[2] HEMBREE, C. H., MELIN, K. B., BOYER, M. C., and SWENSON, H. A. 1950 PROGRESS REPORT OF THE BIGHORN RIVER DRAINAGE BASIN. U.S. Geol. Survey Water Resources Div. [1951.]

FIGURE 2. — Major rivers and reservoirs of the Sacramento and San Joaquin Rivers systems.

TABLE 3.—*Density*[1] *of sediments by material and reservoir operation*[2]

Reservoir operation	Sand		Silt		Clay	
	W	*K*	*W*	*K*	*W*	*K*
Sediment always submerged or nearly submerged	93	0	65	5.7	30	16.0
Normally a moderate reservoir drawdown	93	0	74	2.7	46	10.7
Normally considerable reservoir drawdown	93	0	79	1.0	60	6.0
Reservoir normally empty	93	0	82	.0	78	.0

Example: Sediment in a reservoir, useful life 100 years, with sediment 20 percent sand, 40 percent silt, 40 percent clay. Composite density = 20 percent (93 + 0) + 40 percent (65 + 5.7 $\log_{10}$ 100) + 40 percent (30 + 16 $\log_{10}$ 100) = 93 × 20 percent + 76.4 × 40 percent + 62.0 × 40 percent = 74 lb./cu. ft.

[1] Density is indicated by symbol *W*.
[2] Values for *K* obtained from reference (*11*).

is increased by an amount equal to log time by a constant *K*. Table 3 gives values for *K* under a range of reservoir conditions (*11*).

To determine the average density of sediment inflow delivered to the perimeter of the Delta and the component bays of the system, 58 analyses of particle sizes were studied for distribution in the ranges up to 0.004 mm. for clay, 0.004 to 0.064 mm. for silt, and 0.064 to 0.500 mm. for sand. When unit weights taken from table 3 were assigned for sediment always submerged and any computation factor was omitted, the average unit weight was determined at 48.84 lb. per cubic foot. Table 4 gives the derivation of this figure. Since the California Division of Water Resources arrived at 50 lb./cu. ft. and the USGS, although arriving at a figure of 49 lb. per cu. ft., also adopted 50 lb./cu. ft. as the average density in view of the limited number of analyses concentrated in February 1959 and February 1960, 50 lb./cu. ft. was adopted in this study. Similarly, 90 lb./cu. ft. used by the USGS in this report was adopted for the weight per cubic foot of bedload.

TABLE 4.—*Sediment inflow analysis for San Francisco Bay system*[1]

Number of samples	Month and year	Location	Percent clay	Percent silt		Percent sand		
			<0.004 mm.	0.004 to 0.016 mm.	0.016 to 0.064 mm.	0.064 to 0.125 mm.	0.125 to 0.250 mm.	0.250 to 0.500 mm.
			Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
2	Feb. 1959	San Joaquin at Vernalis	54	30	0	16	.....	.....
3	Feb. 1960	....do	67	13	18	2	.....	.....
7	Feb., Mar. 1959	Sacramento at Sacramento	26	11	17	14	23	9
3	Feb. 1960	....do	38	19	25	7	11	.....
2	Feb. 1959	Yolo Bypass at Woodland	80	16	3	1	.....	.....
2	Feb. 1960	....do	74	19	3	2	1	1
4	Feb. 1959	Alameda Creek near Niles	56	27	12	5	.....	.....
4	Feb. 1960	....do	72	17	9	2	.....	.....
5	Feb. 1959	Napa River near St. Helena	49	15	9	4	11	12
8	Jan.–Mar. 1960	....do	38	20	18	8	16	.....
4	Jan., Feb. 1959	Sonoma Creek, Boyes Hot Springs	29	12	22	13	11	13
5	Jan.–Mar. 1960	....do	45	20	17	8	7	3
3	Feb. 1959	Walnut Creek at Walnut Creek	62	10	19	8	1	.....
2	Feb. 1960	....do	63	19	12	4	2	.....
1	Feb. 1960	Guadalupe at San Jose	92	7	1	.....	.....	.....
3	Jan., Feb. 1960	San Francisquito Creek at Stanford	71	8	17	4	.....	.....
Totals			916	263	202	98	83	38
Percentages			57.2	16.4	12.6	6.2	5.2	2.4
Unit weight—lb. per cubic foot			30	65		93		
Weight per cubic ft. 48.84 (call 50)			17.16	18.85		12.83		

[1] Data abstracted from "Suspended Sediment in California Streams," U.S. Geological Survey, Sacramento, Calif. (Unpublished records.)

These are the average densities of sediment inflow delivered at the meres of the system. Progression downbay toward the Golden Gate results in comminution of the coarser particles and in higher percentages of colloidal particles, extensive flocculation, and, hence, less compaction of sediment with consequent less average sediment densities for suspended load and bedload. If the findings of the USGS study are adopted — the most exhaustive to date — the volume of sediment delivered annually to the Bay system (including the Delta) was 8.8 million cubic yards for the 1957-59 period. Table 5 gives the definitive findings by weight of the USGS investigation.

Direct measurement of the bedload is difficult, due as much to the complexity of the phenomena of bedload movement as to the physical difficulties of measurement, with turbulence be-

TABLE 5.—*Average daily sediment inflow to San Francisco bay system*[1]

Station or stream group	1957–59				1909–59 adjusted to 1957–59 conditions—total sediment
	Suspended sediment		Unmeasured sediment	Total sediment (rounded)	
	measured	estimated			
	Tons	*Tons*	*Tons*	*Tons*	*Tons*
Delta system	12,200	50	1,960	14,200	13,800
Suisun Bay	320	320	60	700	500
San Pablo Bay	218	580	80	900	1,000
San Francisco Bay	983	410	140	1,500	800
Total (rounded)	13,700	1,400	2,200	17,000	16,000

[1] From table 12 of reference (*12*).

ing a principal factor in the movement. Transportation of the bedload is affected by traction manifested either by rolling, sliding, or by particles making small jumps, called "saltation." The width of the stream, permitting either one or more threads of maximum velocity with accompanying areas of maximum turbulence, is also a factor. If the phenomena of formation of dunes and antidunes is also added as factors, it is apparent that, in the present knowledge, the indirect approach to estimation of the bedload is the most practical; that is, measurement of the total load and the subtraction of the suspended load. The USGS investigation determined the total load for some streams at concrete controls or where the channels were bedrock with high turbulence. For other streams with alluvial beds, the unmeasured (bed) load was estimated on the basis of mean velocities and particle size of the bed material. For the major contributors — the Sacramento and San Joaquin Rivers — total sediment discharge was computed (*3*).

TABLE 6.—*1957–59 sediment inflows by geographic areas in percentages by weight and volume*

Geographic area	Suspended load (percent of total for area)		Bedload (percent of total for area)	
	By weight	By volume[1]	By weight	By volume[1]
Delta	86.9	92.26	13.1	7.74
Suisun Bay	91.4	95.02	8.6	4.98
San Pablo Bay	91.1	94.85	8.9	5.15
San Francisco Bay	90.7	94.63	9.3	5.37
Total system (rounded by USGS)	87.0	92.35	13.0	7.65

[1] Conversion based on densities of 90 lb./cu. ft. for bedload and 50 lb./cu. ft. for suspended load.

[3] GRIMM, C. I. STUDY OF TIDAL CURRENTS AND SILT MOVEMENTS IN THE SAN FRANCISCO BAY AREA, WITH PARTICULAR REFERENCE TO THE EFFECT OF A SALT WATER BARRIER UPON THEM. U.S. Corps Engin. Spec. Rpt. (App. D, House Doc. 191, 71st Cong., 3d Sess.) 1931. [Not published.]

Table 5, quoted directly from the USGS investigation (*11*), sets forth the findings from the 1957-59 series of observations and the derived bedload figures (listed as "Unmeasured sediment") by geographical distribution of the sediment inflow. Table 6 gives the percentage distribution of the total sediment load of suspended loads and bedloads by geographical areas. Table 7, derived from tables 5 and 6, gives tonnages and daily sediment inflows by geographical areas and also the rounded annual sediment inflow in cubic yards. In table 7, the 1909-59 figures adjusted to 1957-59 conditions are considered valid for the period 1960–2011.

TABLE 7.—*Sediment inflows to San Francisco Bay system, 1960–2011*

Geographic area	Adjusted total sediment[1]		Daily inflow	Average annual inflow	Percentage of total sediment inflow
	Tons/day	Percentage of total			
			Cu. yd.	*Cu. yd.*	
Delta	13,800	85.7	19,280	7,040,000	85.5
Suisun Bay	500	3.1	720	260,000	3.2
San Pablo Bay	1,000	6.2	1,420	520,000	6.3
San Francisco Bay	800	5.0	1,140	415,000	5.0
Total	16,100	100.0	22,560	8,235,000	100.0

[1] 1909–59 figures adjusted to 1957–59 conditions.

Comparative Estimates

The volume derived from table 5 for the 1957-59 water years agrees almost identically with the volume for the same years derived from a study of historical sedimentation that used Bureau of Reclamation figures for historical outflow of the Sacramento River at Sacramento in the long term period 1924-25 to 1959-60. The total outflow to the Delta for the combined Sacramento-San Joaquin system was determined from a rating curve (fig. 3) giving the relation of the total Sacramento-San Joaquin flow to the flow of the Sacramento River at Sacramento. The sediment-water discharge relation curve (fig. 4), used by the USGS in their 1957-59 study, was then used to obtain the suspended sediment load and this multiplied by 1.2 to obtain total sediment load. Table 8 details the results of this study. The long-term annual average sediment inflow is 6,200,000 cubic yards. For the water years 1957-59, the average is 7,280,000 cubic yards as compared to 7,200,000 given by the U.S. Geological Survey.

Gilbert (*7*) estimated 2 million cubic yards average annual sediment inflow for the period prior to 1849, 18 million for the 1849-1914 period, and 8 million for the future after clearance of the mining debris. Grimm in 1931[3] estimated 5.75 million, and the Corps of Engineers in 1954, assuming considerable reservoir regulation, determined the total at 3.36 million.

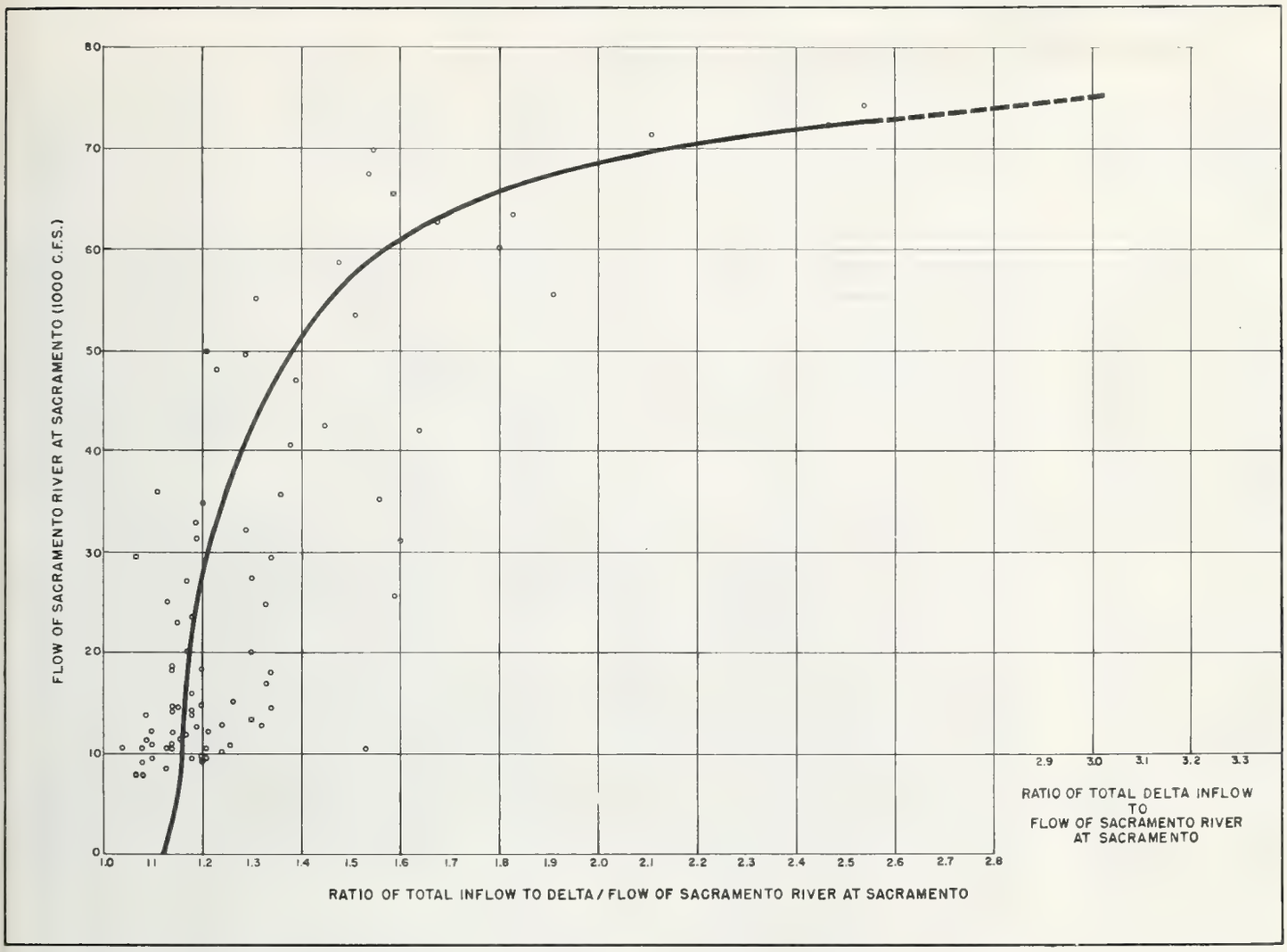

FLOW OF SACRAMENTO RIVER AT SACRAMENTO (1000 C.F.S.)
RATIO OF TOTAL INFLOW TO DELTA / FLOW OF SACRAMENTO RIVER AT SACRAMENTO
RATIO OF TOTAL DELTA INFLOW TO FLOW OF SACRAMENTO RIVER AT SACRAMENTO

FIGURE 3. — Ratio of total delta inflow to flow of Sacramento River at Sacramento.

The State Division of Water Resources,[4] in the 1955 report on the feasibility of barriers, estimated the annual load at 4 million cubic yards. Much of the variance in these figures can be attributed to paucity of data available to the investigators, judgment being a definite factor in the estimating process. The findings of this present study, as noted, are based on records maintained continuously for 3 years.

Factors Affecting Future Sediment Inflow

The estimate of 8,235,000 cubic yards per annum given in table 7 is an average based on appraisals of the major factors affecting the volume of sediment. These are land use, streamflows, and the conditions governing the formation of river channels (river morphology). Rain erosion of the land surface and channel erosion of the streambed effected by the river itself are the main determinants of waterborne sediment. Land use in turn is the major factor in rain erosion, and the competency, capacity, and existing load are the factors determining to what extent, if any, channel erosion and the shaping of new channels will take place.

If the land use aspect of the San Francisco Bay system tributary sediment source areas is considered, the change to urban use will be most marked for the local bay area watersheds and least, relatively inconsequential, for the Sacramento-San Joaquin system. The land use pattern for the local areas in 1960 was 10 percent urban, 60 percent agricultural, and 30 percent mountain, public, and wastelands. By 2020 urban use will approach 35 percent (*6*) (*15*), and agricultural use decline to 40 percent. Since the local drainage areas contribute but 14.5 percent of the present total sediment inflow, these changes will bring no noticeable variations in resulting sediment volume. Within the Sacramento-San Joaquin system the present

[4] CALIFORNIA DIVISION OF WATER RESOURCES. SEDIMENTATION. Appendix G of "Feasibility by the State of Barriers in the San Francisco Bay System." Sept. 1955.

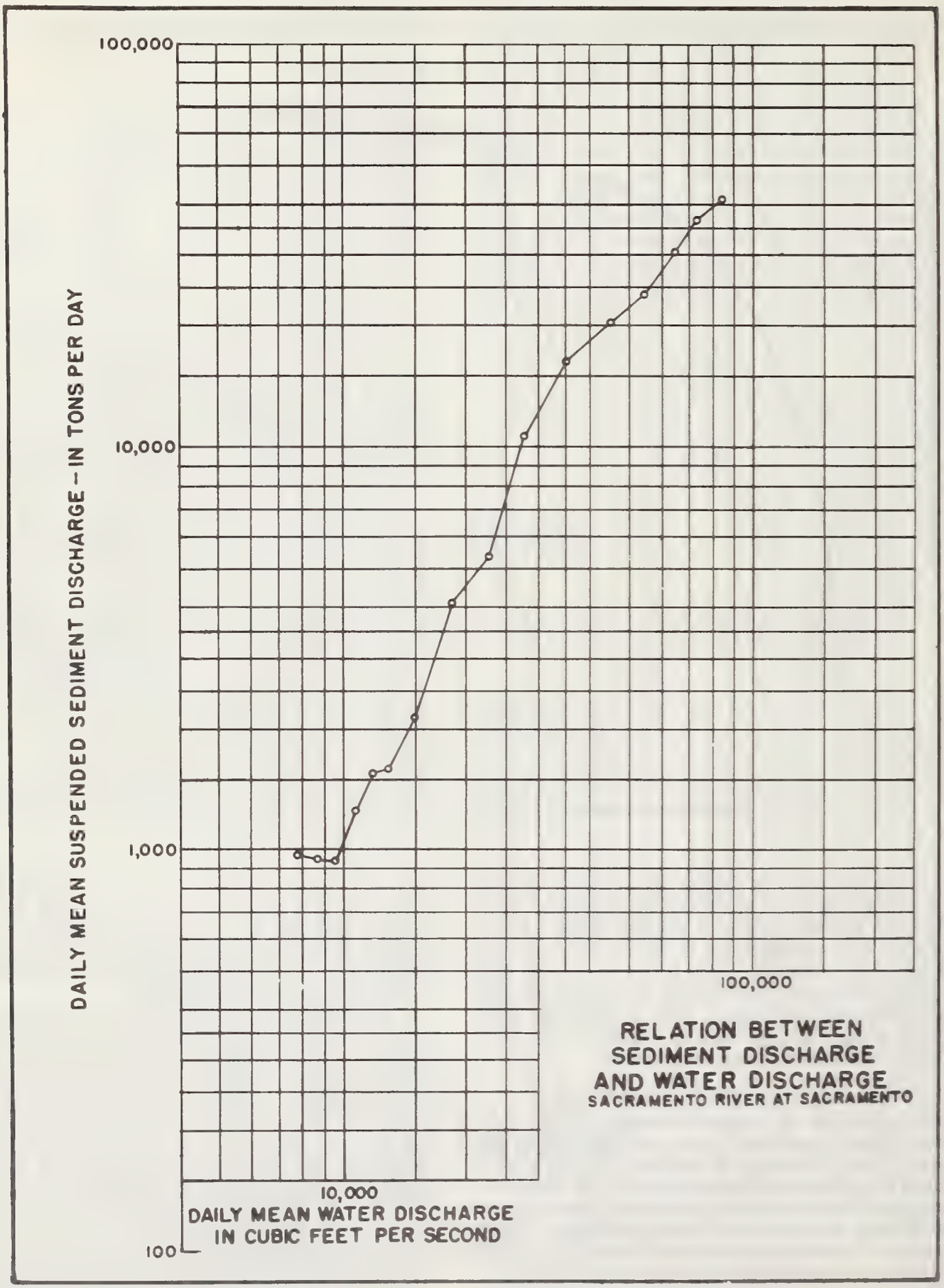


FIGURE 4. — Relation between sediment discharge and water discharge of Sacramento River at Sacramento.

TABLE 8.—*Estimated average monthly historical sediment inflow to the Sacramento-San Joaquin Delta, 1924–60*

(In thousands of cubic yards)

Year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Annual total
1924–25	60	65	99	92	2,010	707	1,298	936	67	61	65	58	5,518
1925–26	59	42	61	76	1,384	353	1,223	153	60	67	67	59	3,604
1926–27	59	683	697	810	2,468	1,554	1,721	1,236	320	57	53	58	9,726
1927–28	59	260	190	301	720	2,146	1,313	368	57	62	65	60	5,601
1928–29	60	51	54	59	222	181	107	193	56	65	65	59	1,172
1929–30	60	41	655	514	478	1,022	548	289	55	65	64	58	3,859
1930–31	59	40	43	68	61	193	59	61	65	70	70	62	851
1931–32	62	41	217	568	668	727	679	1,044	247	62	67	62	4,444
1932–33	63	42	43	59	65	352	352	274	152	67	68	62	1,599
1933–34	61	41	83	502	234	478	196	68	64	68	68	62	1,925
1934–35	62	67	47	809	372	1,113	2,422	1,514	276	61	65	60	6,868
1935–36	59	40	41	1,126	2,001	128	1,362	722	233	61	65	68	5,906
1936–37	60	41	42	46	987	1,396	1,568	1,158	155	62	67	61	5,643
1937–38	59	614	1,760	779	2,514	3,125	2,631	2,497	1,718	78	60	57	15,982
1938–39	58	76	107	97	108	365	208	59	63	70	69	60	1,340
1939–40	61	62	60	1,452	2,103	1,917	1,855	785	88	63	64	60	8,570
1940–41	68	69	784	2,298	2,451	1,933	1,946	1,831	457	63	61	68	12,029
1941–42	68	58	1,135	1,525	2,135	1,024	1,870	1,579	866	58	62	58	10,438
1942–43	57	97	413	1,320	1,486	2,184	1,572	731	1,551	58	68	61	9,598
1943–44	59	56	61	83	470	559	211	349	57	62	62	60	2,089
1944–45	60	93	111	152	1,372	586	572	572	123	57	57	57	3,812
1945–46	59	161	1,184	1,677	365	622	844	617	91	59	59	56	5,794
1946–47	58	92	196	96	339	776	406	76	56	60	60	57	2,272
1947–48	59	85	57	390	106	263	1,590	1,677	1,006	57	58	68	5,416
1948–49	63	78	97	91	94	1,408	1,334	484	64	59	59	59	3,890
1949–50	59	57	59	478	1,251	848	1,101	785	222	59	59	55	5,033
1950–51	65	1,076	3,066	1,643	2,195	1,224	592	662	75	58	61	63	10,780
1951–52	61	105	1,031	2,512	2,664	2,076	3,084	3,556	1,232	232	71	83	16,707
1952–53	63	69	733	2,505	788	491	737	1,085	818	79	58	91	7,517
1953–54	74	105	120	559	1,575	1,428	1,539	463	77	59	59	79	6,137
1954–55	73	122	390	350	122	110	107	307	91	57	57	60	1,846
1955–56	59	63	1,221	5,088	1,731	1,406	847	1,317	492	96	91	68	12,479
1956–57	95	122	95	94	205	1,529	285	817	180	59	61	91	3,633
1957–58	235	245	361	1,044	3,907	2,227	3,886	1,756	1,014	122	118	153	15,068
1958–59	96	96	96	627	1,128	622	107	85	56	89	63	75	3,140
1959–60	58	57	59	81	1,040	954	256	163	75	70	61	59	2,933
36-year total	2,450	5,112	15,478	29,971	41,819	38,117	40,428	30,269	12,279	2,552	2,347	2,387	223,209
36-year average	68	142	430	833	1,161	1,059	1,123	841	341	71	65	66	6,200
Percent by month	1.1	2.3	6.9	13.4	18.7	17.1	18.1	13.6	5.5	1.2	1.0	1.1	100

land use pattern, dominantly agricultural and recreational, will persist with only minor change, percentagewise, in urbanized areas.

Intensified agricultural usage and an expanded highway network will tend to increase the sediment volume, but these tendencies will be offset by improved practices in soil conservation, watershed management, and highway drainage. In 1917 Gilbert (*7*) estimated premining era average sediment inflow at 2 million cubic yards when the region was largely virgin land and forecast 8 million yards for a fully developed highway system through a much expanded agricultural area. The broad conclusion, therefore is that land use changes between 1960 and 2020 will not materially affect the total sediment inflow.

When the future channel erosion and river morphology aspects are considered, the chief additional factors over the natural conditions are the mining debris deposits and the existence of numerous reservoirs. The mining debris load is disposed into the component bays, except for permanent lodgments in the mountains that are stabilized and will not contribute to the sediment inflow. Thus, the appraisal of future loadings narrows down to the effect on competencies and capacities of the rivers of the system from the construction and operation of numerous dams that now ring the tributary area at or about the alluvial fan elevations. The sediments in solution and in the colloid state that would be unaffected by velocity changes could to some extent be altered in content and volume by land usage practices, thus changing the chemistry and the flocculation tendencies.

No such changes would occur to any appreciable extent above the line of reservoirs. Below

the reservoirs the principal agents of such change would be industrial and agricultural contaminants. These in their totality could, with greatly expanded volume, present a disposal problem of large proportions requiring treatment and disposal works. Their effect is not here included, as present knowledge of this phase is inadequate to form quantitative conclusions.

If the preceding feature is omitted, future sediment loads of streams will depend for the most part on conditions below the line of dams. The physical effects of constructing a dam across a stream are pronounced, tending to change the total activity of the river — reshaping its course and affecting the competency, capacity, and load of the current.

As defined by Twenhofel (*14, p. 191*), competency is the ability of currents to transport, in terms of dimensions of particles; capacity is the ability to transport, in terms of quantity; and load is the actual quantity transported. Competency depends upon velocity and turbulence, which in turn vary with the shape of the channel, bottom irregularities, slope, depth, and discharge. There is a competent slope for each variation of channel and for each dimension of particles of a given specific gravity. Competency varies as about the fifth power of the velocity. Capacity is dependent on the same variables as competence, but additional factors are the ratio of depth to width of stream and the degree of mixing of the sediments. Thus, capacity is generally less for material of uniform dimensions than for graded materials. The stream load as a maximum equals the capacity. Capacity ranges between the 3d and

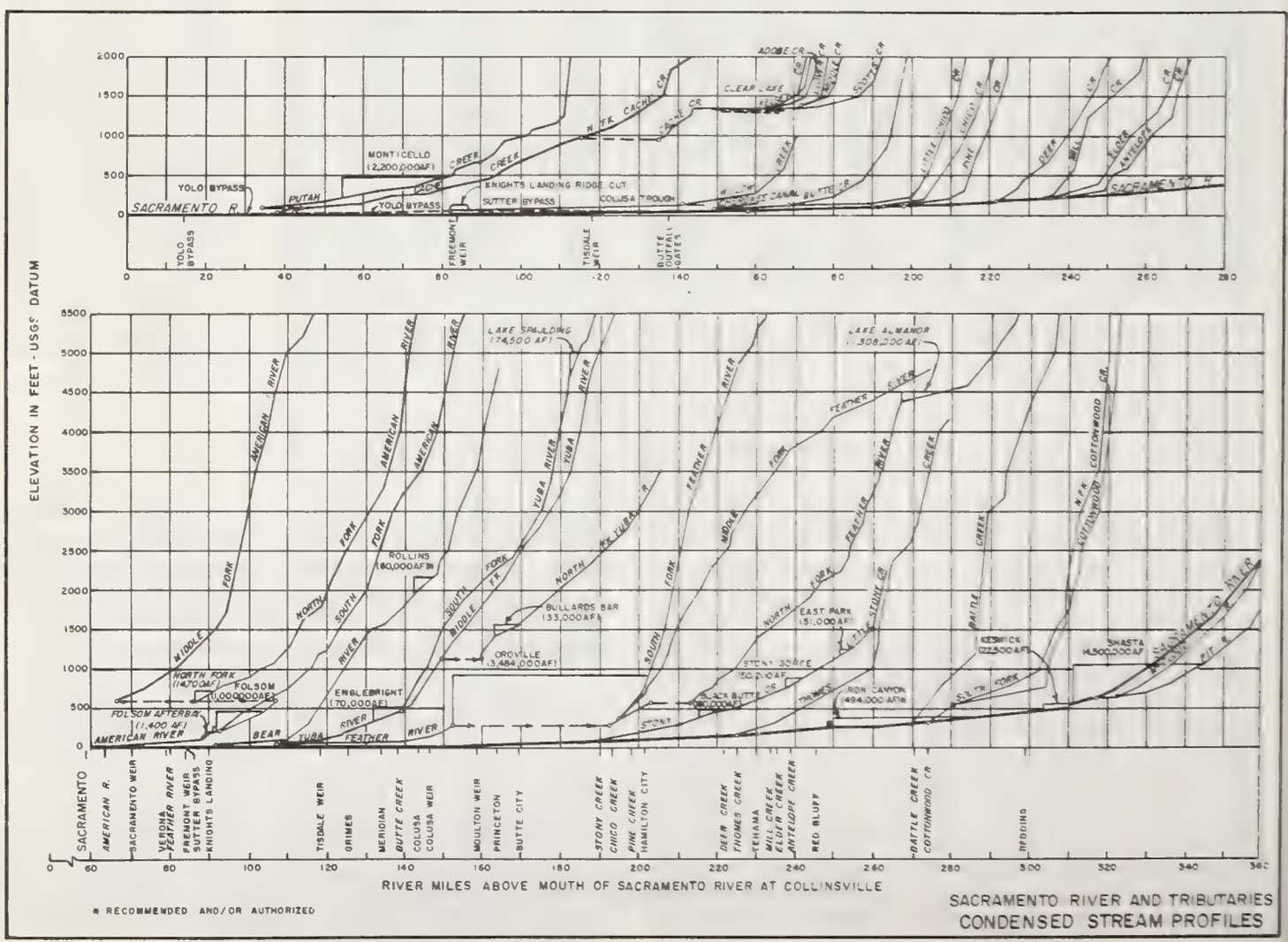


FIGURE 5. — Profiles of elevation and river miles of the Sacramento River and its tributaries.

4th power of the velocity increase. If these considerations to the reaches are applied immediately above and below a dam, the competency upstream is gradually reduced by the widening confines of the reservoir so that a selective grading of deposition results with a tapering of silt at the lower end. Aggradation of the streambed results. With continued aggradation there comes about a loss of cross sectional area, so that scour results and the deposition process takes place further down in the reservoir, the long-run result being complete siltation of the reservoir. With loss of competency and capacity, releases through the reservoir are greatly clarified of suspended load. Virtually all the bedload is usually trapped in the reservoir.

Below a dam, the number and the intensity of the threads of maximum velocity are reduced, when not actually eliminated, by zero flow through the dam or spillway. With reduced turbulence and velocity in the immediate reach below the dam, only the fine silt is picked up in suspension or rolled on the bed in traction. The physical effect on the streambed is loss of fine sediments in the shoal areas and filling of the deeper parts with part of this fine silt load. With flood releases, however, the partly clarified stream under an artificially increased head is characterized by turbulence and high velocity so that scour occurs and degradation ensues. This is the prevalent condition for the Sierra Nevada in the winter flood months.

Table 9 shows that a major part of the total river mileage lies below the line of lowest dams. Further, the dams and reservoirs are in general at the breaks in gradient between the steep upland and the flat sluggish lowland courses where, irrespective of the existence of reservoirs providing stilling basin effects, there would naturally occur heavy deposition of sediment. The corollary follows that the competencies, capacities, and loadings below the dams do not vary greatly from those that prevailed on the below-dam reaches before the reservoirs were constructed. Figure 5 illustrates the location at breaks in grade of the majority of reservoirs constructed or authorized.

It follows, consequently, that the principal effect of the reservoir systems of the Central Valley and local Bay area streams (where the same situation holds) lies in the regulation of flow and thus the regulation of sediment inflow. The competencies and capacities of the various streams would be affected by the velocities and volumes of the regulated flows and, derivatively, the sediment loadings. On analysis, this again is offset — similar to reservoirs at the break in grade — by the concentration of sediment discharge in the flood months. In the long-run

TABLE 9.—*Stream regimens, Sacramento-San Joaquin System*

Stream in system	Elevation at—		Length		Average gradient	
	Source	Lowest dam	Source to lowest dam	Lowest dam to mouth	Upland	Below dam
	Feet	*Feet*	*Miles*	*Miles*	*Feet per 1,000*	*Feet per 1,000*
Sacramento River	8,000	450	64	243	22.4	0.35
Stony Creek	4,000	380	48	24	14.3	2.28
Putah Creek	2,500	185	48	27	10.5	1.11
Feather River	6,000	180	104	64	10.5	0.53
Yuba River	7,500	282	64	44	21.4	1.21
Bear River	3,500	136	38	28	16.87	0.91
American River	8,500	200	81	24	19.4	1.58
Mokleumne River	8,000	230	64	40	23.0	1.09
Calaveras River	6,200	1,600	33	260	1.9	0.21
San Joaquin River	10,000	289	88	230	20.9	0.24
Merced River	11,000	405	72	48	39.0	1.35
Tuolumne River	10,000	331	86	48	21.5	1.19
Stanislaus River	9,000	537	66	48	30.4	2.42
Totals	.....	.....	823	868	.....	.....

average (see table 8), 50 percent of the annual total sediment is delivered in the flood months — January, February, and March. In flood years such as 1958, this figure rises to 70 percent. Thus flood releases would carry a major part of the annual sediment load from the lowland reaches and a certain amount of reservoir-trapped sediment through scour effect. At 7 ft./sec. velocity, all but the heaviest stones would be moved by scour. At 3½ ft./sec. (*14, p. 193*), pebbles up to walnut size would be scoured. Hence, since a flood release velocity of 3½ ft./sec. would prevail, it can be taken the sediment load below a dam would be at the maximum competence and capacity of the stream.

For the remaining months, the average conditions of flow volume and velocity would, as regards sediment inflow, about balance the total under natural conditions ranging from moderate to almost zero flow. Hence, the overall conditions for effects of reservoir at or near the alluvial fan elevation in the tributary sediment area to the Delta would seem to warrant the conclusion that no marked variation from the estimated annual average sediment figure of 7,040,000 cubic yards will occur.

The sediment inflow to the Delta will be reduced in its passage to Suisun Bay by deposition within the sloughs and by an amount carried by the transfer flows exported through the Tracy

pumping plant and the Contra Costa Canal.[5] The estimate of 7.04 million cubic yards of inflow to the Delta, based on the 1957-59 observations, covers withdrawal conditions above the Delta as of 1960. Reduction in sediment volume for future increased withdrawals and for the transfer flows to the Contra Costa Canal and through the Tracy pumping plant will closely approximate the suspended load. The bedload for the greater part will remain in the system. If the water-sediment discharge relation curve (fig. 4) is used, the reduction in suspended load for withdrawals can be obtained. Figure 4 is considered applicable to withdrawals over the entire system, including volumes from local bay area streams. Withdrawals within the Delta totaled 4,500 c.f.s. in 1960, the corresponding sediment withdrawal being 600 tons per annum, or 307,000 cubic yards per annum. Table 10

TABLE 10.—*Sediment in flows to Bay system adjusted for varying water exports before deposition in Delta*
(Total sediment 1960 inflow of 8,235,000 cu. yd.)

Water volume exported (c.f.s.)	Sediment reduction	Reduction[1]	Net sediment inflow[1] per annum
	Tons	*Cu. yd.*	*Cu. yd.*
4,500	600	307,000	7,928,000
10,000	1,000	511,000	7,724,000
15,000	2,200	1,124,200	7,110,800
20,000	4,500	2,299,500	5,935,500

[1] Suspended load. Does not include bedload.

gives the range of sediment inflow for future transfers before accounting for deposition in the Delta.

Sedimentation Within the System

The estimated average annual inflow of 8,235,000 cubic yards will be delivered to the entire system and will be distributed to the component elements in the proportions given in table 7, the Delta receiving 7,040,000 cubic yards, or 85.5 percent. At present, the transfer volumes exported from the Delta average 4,500 c.f.s., representing 307,000 cubic yards of suspended sediment per annum. The sediment inflow to Suisun Bay, therefore, is 7,133,000 yards, less deposition within the Delta.

Determination of the amount of sediment deposited in the Delta — as with the other parts of the system — presents difficulties in view of the complex conditions that affect deposition, such as flocculation and tidal currents. Although research has been productive qualitatively in the field of estuarine sedimentation, a methodology derived from the mechanics of the phenomena involved that would give quantitative as well as qualitative results has not yet been perfected. Hence, an indirect approach becomes necessary to estimate present rates of deposition.

The lower reaches of the Delta are subject to cyclic flocculation as the fresh-saline water interface fluctuates upstream and downstream with the tides and all reaches are subject to the tidal impulses that tend to a storage effect on floodtide when the river inflow is met by the oncoming tidal rise, thus favoring deposition. The emptying effect of the ebbtide, particularly the long ebb, tends to cause resuspension of sediment and to scour. Similarly the advance and recedence of the saline interface brings flocculation of colloidal matter and later resuspension in the colloidal state when fresh water conditions occur. In all instances, because of the predominance of the ebb flow, a residual downstream movement of deposited material takes place. Comparison, therefore, at regular intervals over an extended period indicate changes in channel configurations throughout the Delta that would afford quantitative data on the deposition processes. In the light of qualitative research findings, reasonable projections into the future could possibly be made. Unfortunately, no regular sequential data over a long period are available for the Delta. Hence, as the best available alternate, the dredging records for the Sacramento River, the San Joaquin River, and Old River Slough, which are continuous for the 30-year period, 1930–59, are adopted.

In using these records a primary consideration is to what extent, if any, the diminishing volumes of debris from the hydraulic mining era are represented in the figures given in table 11.

TABLE 11.—*Dredging of Sacramento-San Joaquin Delta, 1930–59*[1]

Years	Maintenance	New work	Total
	1,000 cu. yd.	*1,000 cu. yd.*	*1,000 cu. yd.*
1930	1,055	906	1,961
1931–35	5,420	24,174	29,594
1936–40	12,380	7,469	19,849
1941–45	7,826	.....	7,826
1946–50	6,223	280	6,503
1951–55	4,554	5,269	9,823
1956–59	2,533	.....	2,533
Totals	39,991	38,098	78,089
30-year average	1,333	1,270	2,603

[1] Data from files of U. S. Army Engineer District, Sacramento.

Gilbert (7) estimated that in 50 years from 1914 all debris except permanent lodgments in the mountains would have been cleared from

[5] At Tracy, Calif., is located the pumping plant that supplies the Delta-Mendota Canal, the supply being pumped from Old River. Since the Sacramento River furnishes the major part of the supply, there is a cross-delta transfer of flow that otherwise would follow the course of the Sacramento River into Suisun Bay.

the Bay system. This estimate has been borne out by subsequent developments. At the present time (1962), observations indicate stabilization has come about. The enormous quantities involved in this movement, estimated by Gilbert at 850 million cubic yards, of which 375 million would be sand and gravel subject to deposition at reduced velocities, preclude the possibility that the Delta could be the depository without far greater volumes of dredging than those given in table 11 — 375 million cubic yards that would be deposited as against 78 million dredged in 30 years. The conclusion must be made that the great bulk of this material has been swept through the Delta on into the Bay system. The continuous development of channel improvements for expeditious removal of floodflows lends strength to this viewpoint — that in effect all debris has been cleared through the Delta except minor quantities deposited in backwaters.

Deposits in the Delta, therefore, can be taken as the normal sedimentation for years of medium to low flows, resulting on the slackening of velocity in the Delta sloughs and residual flocculation at the western end of the Delta. This hypothesis assumes the mining debris load and heavy channel erosion load were removed by floodflows occurring at least at 5-year intervals and that the maintenance dredging average of 1,333,000 cubic yards represents dry low-flow years accumulations plus aftermath lodgments from the diminishing flood flows. If 85 percent represents the total Delta deposition (the minor sloughs not being dredged), the average annual sediment deposition in the Delta is estimated at 1.6 million cubic yards.

Sediment Inflow Distribution

From the preceding discussion, the direct sediment inflow into the Bay system, excluding the Delta, i.e., deducting sediment exported with the transfer volume of 4,500 c.f.s. and 1.6 million cubic yards deposition within the Delta, is given in table 12. These sediment volumes — part in the colloidal state, part in suspension, and part being moved by traction (bedload) — upon delivery into the tidal waters of the system enter an environment of higher salinity, cyclic reversals of flow and much greater wind action over large expanses of shallow water. In the changed environment the sedimentation processes are greatly changed, the end result being comminution of particles and a higher flocculation precipitation. These changes are reflected in density changes in the bottom sediments and elimination of the sand content by reduction into silt and colloids.

TABLE 12.—*Direct sediment inflows (1960 conditions) to Bay system*

Bay	Average annual sediment inflow
	Cubic yards
Suisun Bay (from delta)	5,133,000
Suisun Bay (from local)	260,000
San Pablo Bay	520,000
San Francisco Bay	415,000
Total	6,328,000

The changes effected in the character of the sediment load upon entrance to the tidal system and subjection to the changed environment vary over the limits of the component bays. However, a general indication is obtained by comparing the data given in table 4 with the data presented in table 13 compiled from 40 analysis of suspended sediment load taken from the Bay in September 1957 (*13*). The sand content is eliminated in the Bay suspended sediment load and the silt content increased, as shown in table 14.

Bottom sediment densities, which compare relatively with bedload densities of the inflow sediment load to the Delta, while varying by location within the Bay system, are consistently lighter than the 90 lb. per cu. ft. determined by the USGS for the Delta bedload; the differences are shown in table 15.

Other major aspects affecting sedimentation characteristics and deposition are salinity, turbidity, and wind effects. Table 16, giving salinity and turbidity (sediment p.p.m.), is to be considered in a relatively locational sense only, since wide ranges in values occur at every location due to the tidal cycles varying over the year and the seasonal fluctuation being pronounced. The data presented are averages of hourly readings made 21-22 September 1956 by the U.S. Army Engineer District, San Francisco. Within each flood and ebb cycle the sediment (parts per million) fluctuates from lows at beginning and end to maxima in the middle of the cycle, the maximum being approximately double the minimum value. Salinity shows considerably less variation and in the lower reaches none. Temperature and wind effects are discussed later in connection with historical patterns of deposition.

The environmental changes brought about as the sediment inflows from the river systems traverse the bay can be broadly grouped into (1) changes in characteristics and (2) changes in deposition patterns. From the preceding data the principal changes in characteristics are the elimination of the sand content, increase in silt content, and decreased densities of bottom sediment regarded as equivalent to the bedload of the fluvial sedimentation conditions. It is significant that no change occurs in the clay content percentage of the suspended load char-

TABLE 13.—*Bay sediment analyses (suspended load), September 1957*

Field samples		Clay	Silt		Sand		
Sample No.	Depth	Less than 0.004 mm.	0.004 to 0.016 mm.	0.016 to 0.064 mm.	0.065 to 0.125 mm.	0.125 to 0.250 mm.	0.250 to 0.500 mm.
	Feet	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
8T	27.5	54	45	1	.....	.....	.....
16T	26.5	47	50	3	.....	.....	.....
24	.....	50	48	2	.....	.....	.....
40T	27.5	52	46	2	.....	.....	.....
48T	.....	48	51	1	.....	.....	.....
56T	25.5	48	50	2	.....	.....	.....
55T	25.5	74	24	2	.....	.....	.....
57T	26.5	50	48	2	.....	.....	.....
58T	25.5	57	43	.....	.....	.....	.....
69T	25.0	51	47	2	.....	.....	.....
70T	25.5	49	49	2	.....	.....	.....
80	28.5	65	32	3	.....	.....	.....
89	29.0	44	56	.....	.....	.....	.....
103	31.5	55	45	.....	.....	.....	.....
111	31.5	58	38	4	.....	.....	.....
123	29.0	50	47	3	.....	.....	.....
128	27.5	47	50	3	.....	.....	.....
135	27.0	63	35	2	.....	.....	.....
143	26.0	85	15	.....	.....	.....	.....
169	30.0	45	52	3	.....	.....	.....
170	30.5	40	57	3	.....	.....	.....
182	30.5	65	35	.....	.....	.....	.....
189	30.5	72	27	1	.....	.....	.....
197	30.5	53	45	2	.....	.....	.....
204	30.5	46	53	1	.....	.....	.....
211	29.5	62	38	.....	.....	.....	.....
224	28.5	48	52	.....	.....	.....	.....
230	27.5	60	40	.....	.....	.....	.....
7 [1]	26.5 to 30	[1]48	[1]42	[1]10	.....	.....	.....
5 [2]	27 to 31	[2]79	[2]19	[2]2	.....	.....	.....
Average	.....	57	40	3	.....	.....	.....
Unit weight	.....	30	65		.....	93	.....
Weight [3]	.....	19.1	27.95		.....	.....	.....

[1] 7 samples; multiply percentages by 7.
[2] 5 samples; multiply percentages by 5.
[3] Weight per cubic foot is 47.05 lb., or 47.

acteristics — remaining at 57 percent. The deposition patterns are changed from fluvial types as determined by competency, capacity, and loading of the carrier streams, to fluctuating tidal forward and backward movements modified by salinity changes, wind effects, and bottom currents.

Increase in Volume

Of practical consequence is the increase in volume resulting from the lowered densities of Bay bottom sediment in comparison with the bedload densities under fluvial conditions. The significant feature — the percentage of clay content in the suspended load remains constant

TABLE 14.—*Change in content of suspended loads, Delta sediment inflow and Bay sediment*

Source	Clay	Silt	Sand
	Pct.	*Pct.*	*Pct.*
Delta inflow	57	29	14
Bay system	57	43	0
Change (to Bay system)	0	+14	−14

under fluvial and Bay tidal conditions — would indicate that flocculation, which removes a high percentage of colloids and tends to increase the bedload, is balanced by the addition of colloids resulting from attrition of the coarser particles by tidal action and turbulence, both tending to reduce in the bedload and to increase in the suspended load. In this connection, no material change occurs in the weight per cubic foot of suspended sediment for fluvial and bay conditions — 48.84 lb. per cu. ft. for fluvial sediment and 47.05 lb. per cu. ft. for Bay suspended sediment. Hence, it is concluded there is no great

TABLE 15.—*Bottom sediment densities*[1] *San Francisco Bay system*

General location	Bottom sediment density
	Lb./cu. ft.
Suisun Bay	88
Central (North San Francisco) Bay	49
South San Francisco Bay	45
San Pablo Bay	31

[1] Analysis of 124 dredge spoil samples over period 1953–62 U.S. Army Engineers, San Francisco District Operations Division.

material change in proportions of total sediment by weight per cubic foot between suspended load and bedload.

From this reasoning increase in volume is proportional to the changes in densities of the suspended loads and bottom sediments as follows:

	Percent
Suisun Bay	6.0
San Pablo Bay	24.6
San Francisco Bay	12.6

The bulk of the increase is actually concentrated in the bedload to bottom sediment increase, in part accounting for the large shoaling volumes

TABLE 16.—*Average salinity and sediment throughout the San Francisco Bay system, Sept. 21–22, 1956*

Location	Salinity [1]		Sediment	
	Flood	Ebb	Flood	Ebb
	P.p.t.	*P.p.t.*	*P.p.m.*	*P.p.m.*
Collinsville	0.7	0.7	89	71
Carquinez Strait	19.6	17.1	76	80
San Pablo Strait	24.3	.....	12	.....
East of Treasure Island	29.9	29.9	27	23
Dumbarton	29.0	29.0	97	48

[1] In parts per thousand.

occurring in the component bays. As noted previously these increases for bedload to bottom sediment are 2 percent for Suisun Bay, 249 percent for San Pablo Bay, and an average of 91 percent for San Francisco Bay. A cubic yard of bedload material, e.g., becomes 3½ cubic yards of bottom sediment in San Pablo Bay.

Deposition of Sediment in the Tidal Bays

Upon entrance into the Bay system, the vehicle of carriage for the fluvial sediment becomes the tidal prism, which is characterized by alternate advances and recessions so that net movement downbay is very much less than under the fluvial movement in a single direction —downstream. The tidal prism is averaged at 1.575 million acre-feet, or 788,000 c.f.s., and if the average daily fresh water inflow is set at 29,000 c.f.s., or 14,500 c.f.s. for a complete 12-hour cycle, the dilution ratio is approximately 54:1. The total volume of the Bay system, however, is much greater than the tidal prism, and approximates 5.6 million acre-feet below mean sea level, excluding the Delta, which permits still greater dilution. However, there is no marked difference between the sediment concentrations under fluvial and under Bay tidal conditions, indicating that supplementary supplies are being made to the suspended load in the Bay waters either from outside the system from the ocean or from bottom mud stirred into suspension by tidal action.

Krone, in his studies on estuarial shoaling in the Bay system (*10*), states with regard to shoal material "most of the shoal material appeared to have originated from inland drainage by way of the San Joaquin-Sacramento River system, which carried principally montmorillonite, and less from coast range drainage, which carried principally illite. It was concluded that sediment is mixed throughout the Bay system, and that the composition and particle-size distribution resulted from the combination of sediment loads from the contiguous streams, as modified by local transport conditions." This statement would favor the viewpoint of the supplemental supply originating from bottom deposits, since an ocean supply would necessarily be reflected in the analysis of bottom sediment. This is supported by the general functioning of the system whereby a net daily outflow constantly occurs, entailing sediment outflow to rather sediment inflow from the ocean. It is concluded, therefore, that the additional sediment load in the tidal waters over that being delivered from the fluvial sediment input is derived from the bay bottom sediments and is a result of tidal and wind action.

The pattern of deposition, consequently, comes from the net translation and deposition resulting from reiterated reversals of competencies and capacities of the tidal flows modified by salinity variations and wind effects. The pattern is in continuous flux, a basic daily pattern being adjusted to an overriding seasonal fluctuation and a long-term flood-drought cyclical swing modifying these basic fluctuations. Geologic action also affects the deposition of sediment, such as differential settlement of land in different locations of the system. A slow rise of the Pacific Ocean is also known to be occurring. Thus as a result of the superimposed long term fluctuations over a number of decades, parts of the system would be slowly changing from a condition of accretion to one of scour or vice versa.

The normal pattern for fluvial sediment entering the tidal system is for a net daily progression downbay of the bedload increased by flocculation as the salinity concentration increases. The net downbay movement is evident on comparison of the competencies and capacities of the flood and ebbtides through a complete daily cycle for a mixed tide. The net downbay movement is not pronounced for a typical sine tide, and for the recurring case of floodtide dominance net movement would be upstream. Over the long run, and particularly for the flood months of heavy sediment inflow, net movement is downbay. If the data for a mixed tide were analyzed, an index of the overall movement downbay is obtained. For the tides of November 11-12, 1958, the average of the velocities of the two ebbtides exceeded that of the two floods by 7 percent and the corresponding average tidal prism for the ebbs exceeded that for the floods by 2.7 percent. Thus, the competency of the ebbs is 40 percent greater than the flood competency. Hence, with the greater carrying volume of the ebbtides, the resulting capacity of the ebbtides exceeded by 30 percent that of the floodtides. However, the dominance of the flood duration over the ebb duration mitigates the effect of the ebb capacity dominance, thus cutting down the net movement downbay of the suspended particles to about 0.4 mile. For sediment on the bottom, part fluvial bedload and part eroded from the bottom deposits, many variables enter into the net movement that takes place. For this aspect the movement and volume of deposits and the rates of accretion and scour recourse are best for analysis of changes in bottom configurations.

The Historical Sedimentation Pattern

For the Bay system changes in bottom configurations can be ascertained from the soundings surveys conducted at periodic intervals from 1855 through 1956. The changes thus ob-

tained give the volumes of accretion or scour for the time intervals noted, but the changes do not give the total net deposition that has taken place throughout geologic time. This total was approximated through a series of borings from which contours of the bottom of the Bay mud lying on firm clay or sand were established. The contours for the top of the Bay mud were then obtained from the most recent soundings; by subtraction, the net volumes of Bay mud were derived, the total being slightly under 16 billion cubic yards. Table 17 details the deposi-

TABLE 17.—*Bay Mud volumes, San Francisco Bay system*

Location	Water area	Average depth to clays and sands	Volume to clays and sands	Water volume	Bay mud volume	
	Acres	*Feet*	*1,000 acre-feet*	*1,000 acre-feet*	*1,000 acre-feet*	*Millions of cubic yards*
Suisun Bay:						
Army Point to Chipps Island	22,967	29.8	684	268	416	671
Carquinez Strait:						
Napa River to Army Point	4,350	74.0	322	172	150	243
San Pablo Bay:						
Point San Pablo to Napa River	58,361	57.3	3,344	631	2,713	4,379
Central San Francisco Bay:						
Golden Gate to south end, Yerba Buena Island to Point San Pueblo	52,968	92.0	4,873	1,897	2,976	4,801
San Francisco Narrows:						
South end of Yerba Buena Island to Candlestick Pt.	21,255	62.0	1,318	592	726	1,170
South Bay:						
Candlestick Point to Alviso	83,950	46.0	3,862	951	2,911	4,695
Total: Bay system	243,851	[1]59.06	14,403	4,511	9,892	15,959

[1] Average.

tion of the Bay mud deposits in the component bays. Deposits are lightest in Suisun Bay, heaviest in Central Bay, and roughly equal for the remaining areas, the ratio of deposition per acre being 1:3:2 for Suisun Bay, the Central Bay, and the approximately equal (by rate per acre) areas — Carquinez Strait, San Pablo Bay, and South Bay.

The total of 16 billion yards of deposits of bay mud is the net result of an indeterminate period extending back into geologic time. The present water volume of the system is slightly in excess of 10 billion cubic yards, but assumption, based on rough comparison with the 16 billion yards deposit, that a very great period would have to elapse for complete filling of all bays is not warranted. With increasing reduction of the tidal prism, the San Francisco bar would move landward with probable inflow of ocean sediment. Accelerated deposition with augmented volume of sediment would shorten the period of ultimate fill. Net deposition per annum determined from bottom configuration changes over a 100-year period is 6 million yards. If this rate would remain constant, the period for complete siltation of the system would be 1,700 years.

From the record, however, cycles of erosion and accretion occur that make unreliable any straight line projection for the pattern of deposition. A close analysis of these cyclical trends, however, is difficult, because of the overlapping and lack of continuity in the basic source data. Also some survey data have been questioned for reliability by previous investigations. For this study, data and findings of previous studies were appraised in the light of recent observations.

The source data are the surveys conducted by the USC&GS from 1855 at intervals through 1956. The investigations consulted were (1) those by Gilbert (*7*), completed in 1914; (2) the comprehensive study by the U.S. Army Corps of Engineers, known as the Grimm study completed in 1930 but not published; and (3) the Division of Water Resources, State of California, investigation released in March 1955 as part of the report "Feasibility of Construction by the State of Barriers in the San Francisco Bay System." The first comprehensive surveys by the USC&GS were carried out between 1855 and 1868. Gilbert questioned the reliability of the vertical controls used for the surveys of the northern half of San Francisco Bay and of San Pablo Bay. The USC&GS in rebuttal countered that such vertical controls were relatively unimportant, as compared to the accuracy of the depth measurements. For the southern half of San Francisco Bay (1857-58) and for Suisun Bay (1867-68), local tide stations, some tied into bench marks, supplemented the permanent station at Fort Point. The Grimm study supplemented the source data with surveys of Suisun and San Pablo Bays in 1930, but the fieldbooks and boat sheets have not been located. Changes in shorelines between surveys also made difficult the computation of volume changes on a bank to bank basis.

The methods of using the source data affect the results. Volume changes can be determined by establishing bottom contours, planimetering the areas, and computing volumes by depth intervals. An alternate method is to establish a grid over the system and average the depth of the recorded soundings within the quadrilateral; the area by depth giving the water volume when compared with the altered water volume of a subsequent survey determines the volume change in deposition. The accuracy of the grid method depends on the uniformity of spacing of the depth measurements within the quadrilateral, the shape of the bottom, and the area. The accuracy of the contour method depends on the horizontal accuracy of the contour location. Gilbert apparently used both methods, using either where most adaptable. Grimm used one minute (longitude and latitude) quadrangles, but used contours to some extent as subdivisions of the main tract. The State adopted ¼-minute quadrangles together with cross sections, to develop better the fringe tracts, especially. The present Corps of Engineers study used ⅛-minute quadrangles.

If it is assumed that water volume change equals sediment volume change, the possibility of subsidence of the Bay floors is discounted. This possibility, for lack of data to evaluate properly, was discounted in all prior investigations. In this study the possibility of subsidence was evaluated from data furnished by the U.S. Geological Survey (*8*) covering subsidence in Santa Clara County. The effect is relatively minor and is not included in the findings graphed in figures 6 through 14, which give cumulative sediment volume changes throughout the system. With this feature discounted, the sedimentation pattern is given by the major component areas.

Suisun Bay

Figure 7 gives the sedimentation changes in 1-minute quadrangles and table 18 gives the changes by major areas for Suisun Bay. Excepting the channels that range in depth from 20 to 70 feet (including a series of navigation channels with authorized maintenance depth of 30 feet and width of 300 feet), Suisun Bay is markedly shallow, generally less than 6 feet in depth. For the period of record, the main deposition took place in the 21-year period 1867-86 and was laid down chiefly in the channel areas. By 1921 scour had reduced the 22.5 million yards to 7.3 million and to actual scour by 1942 despite a temporary accretion recorded in 1930. The area covered by the West Bay area and Suisun slough channel experienced the same general depletion trend. Similarly and to a marked degree with Honker Bay, the pattern changed from a deposit of 2.7 million yards by 1886 to a scour of 11.8 million by 1942. The Grizzly Bay area was relatively inactive through the 90-year period.

TABLE 18.—*Cumulative sediment volume changes, Suisun Bay 1866–1956*

Area	1866–1887	1921	1930	1941–42	1956
	Million cu. yd.	*Million cu. yd.*	*Million cu. yd.*	*Million cu. yd.*	*Million cu. yd.*
Channel areas and middle ground	22.5	7.3	11.5	−0.1	.....
West Bay and Suisun slough channel	10.0	3.2	5.7	5.5	.....
Honker bay	2.7	−9.5	−6.3	−11.8	.....
Grizzly Bay	3.2	.....	.....	.....	1.8
Totals	38.4	1.0	10.9	−6.4	.....

Carquinez Strait

The accumulated sediment volume change for Carquinez Strait is given in table 19 and figure 8. The strait, in general a mile in width between steep escarpments and 8 miles long, is deepest in the middle (Dillon Point) with soundings to 100 feet and much shallower depths at either end — about 30 feet.

TABLE 19.—*Cumulative sediment volume change, Carquinez Straight, 1856–1950*

Years	Volume
	Million cubic yards
1856–86	70.0
1922	81.0
1950	98.3

An overall accretion of 98.3 million yards, the greater part during the years 1857-86, is shown, but this masks areas of scour and fill occurring within the same quadrangle. This particularly applies to two quadrilaterals south of Benicia bounded by 122° 8′ W. to 122° 10′ W. and 38° 2′ N. to 38° 3′ N. Detailed study of these areas showed very heavy sedimentation occurred along the north shore of the strait accompanied by compensating scour in the channel area. The mean lower low water line of Southampton Bay had advanced about 2,500 feet from deep inside the Bay to a part alined with the sweep of the strait. The mean level low water line is now located where water depths were 90 feet in 1857, most of the filling occurring in the 1857–86 period. In the vicinity of Benicia Arsenal Pier a shore line advance of 500 to 700 feet occurred during 1857 to 1886, after which the situation became stabilized until 1942. Accentuated shoaling is indicated in the vicinity of the new arsenal dock (built in 1941-42) from the data furnished by the 1949 survey.

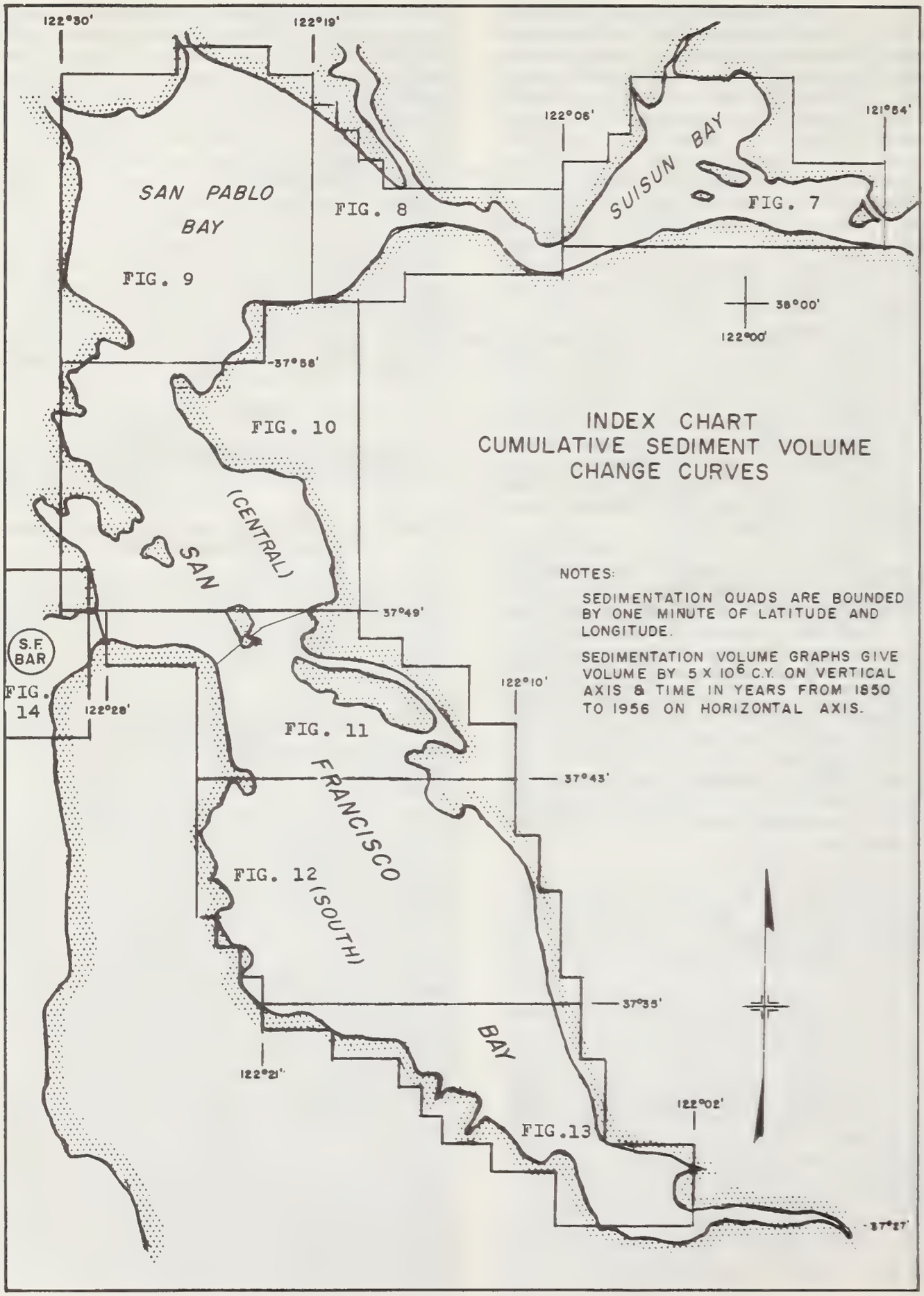


FIGURE 6. — Index chart of cumulative sediment volume change curves of San Francisco Bay. (See figs. 7 to 14.)

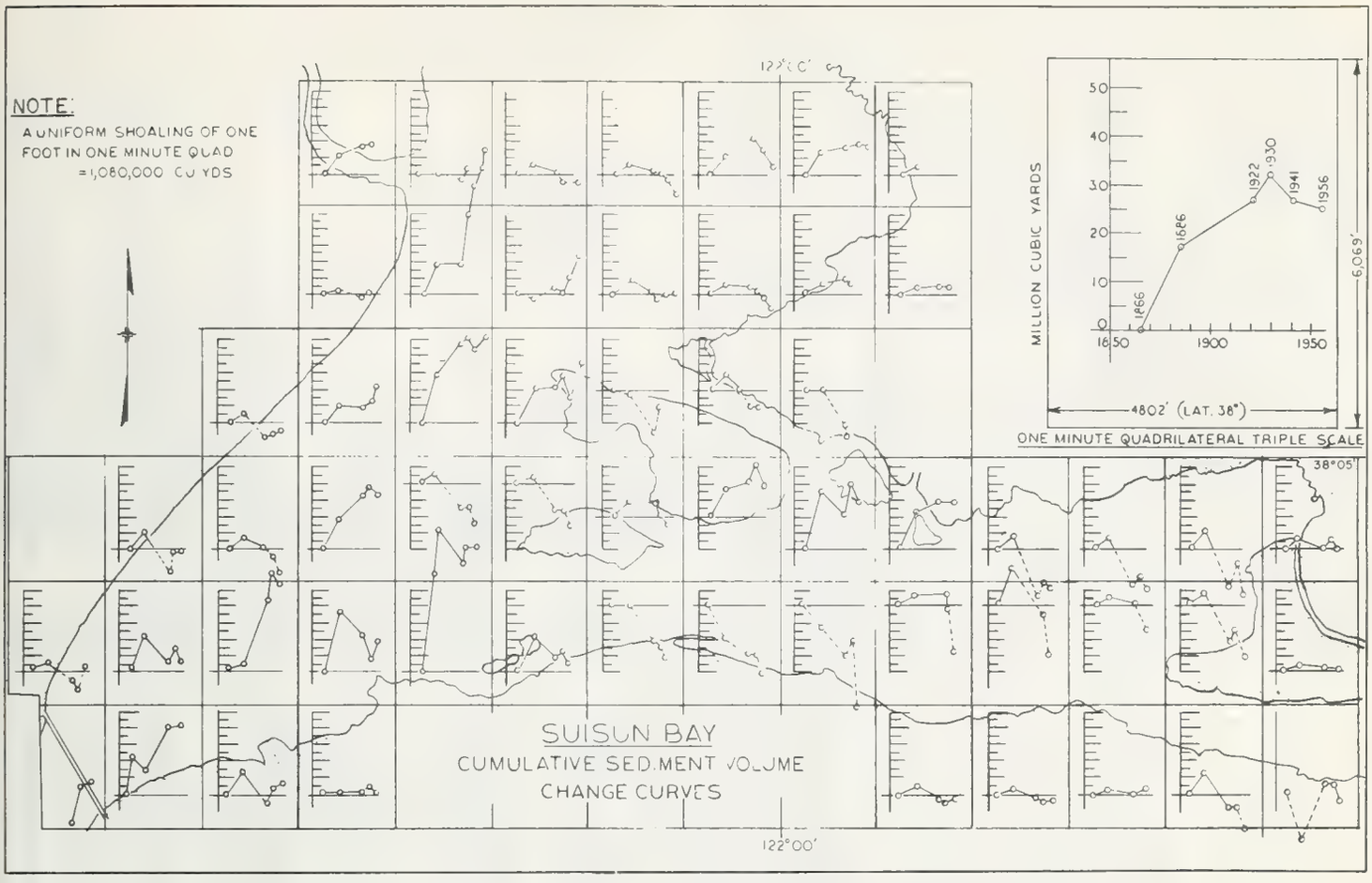


FIGURE 7. — Cumulative sediment volume change curves of Suisan Bay.

San Pablo Bay

The sedimentation pattern of San Pablo Bay was studied by three subdivisions according to depth: (1) shoal areas of 4 feet depth or less extending over 39 quadrilaterals, (2) shoals between 4 and 20 feet in depth covering 40 quadrilaterals, and (3) channel areas over 20 feet deep covering 20½ quadrilaterals. Table 20 and figure 9 give the sediment change volume data by periods. The 4 feet and less shoal areas lie mostly in the northern and western limits of the Bay, although small areas contiguous to the southern shoreline are included. (See fig. 17). The highest deposition, relatively, has occurred in the shallow areas, with the major part occurring in 1856-87 period. The intermediate depth areas, 4 to 20 feet, lie on either side of the channel areas with the greater extent north of the channels. These areas for the total period show over twice the deposition of the marginal shoals and over 4 times that of the channel areas. The channel areas are conspicuous by consistently showing scour from the initial period of deposition in 1856-87.

North San Francisco Bay and Golden Gate

North San Francisco Bay, regarded in this paper as the central bay between the northern and western arm (San Pablo and Suisun Bays) and the southern arm (below Yerba Buena) of the Bay system, is analyzed on a similar basis to San Pablo Bay by considering marginal shoals (0 to 10 feet), intermediate shoals (10 to 30 feet), and channel areas (over 30 feet). The

TABLE 20.—*Cumulative sediment and mean depth changes, San Pablo Bay, 1855–1951*

Area	1885–87	1899	1922	1930	1951
Marginal shoal (0 to 4 feet) areas, million cubic yards	74.0	75.8	101.2	76.3	105.3
Mean depth of deposits, feet	1.80	1.81	2.40	1.81	2.50
Intermediate (4 to 20 feet) shoal, million cubic yards	162.0	169.9	214.8	177.1	251.6
Mean areas depth of deposits, feet	3.74	3.92	4.95	4.10	5.80
Channel (20+ feet) areas, million cubic yards	88.4	83.6	74.4	47.4	56.3
Mean depth of deposits, feet	3.98	3.77	3.35	2.14	2.54
Total volume, million cubic yards	324.0	329.3	390.4	300.8	413.2
Total average depth, feet	3.00	3.04	3.60	2.78	3.80

0- to 10-foot shoals are the Berkeley mud flats and the drowned valleys of the western shore covering 22.9 quadrilaterals. The intermediate depths flank the deep water and include Southampton shoal. These areas cover 18.2 quadrilaterals. The channel areas cover 26.9 quadrilaterals within the central part of the total area and lead into Golden Gate, 6 quadrilaterals. Figures 10 and 14 and table 21 summarize the volume and depth changes from 1855 to 1948 for North San Francisco Bay and through 1956 for the Golden Gate.

The marginal shoals of the Bay eroded 0.58 foot in the initial 40-year period and then reversed to accretion in relatively small amounts that were insufficient to overcome the previous erosion: the overall effect from 1855 to 1948 being a net erosion of 0.14 foot. The intermediate shoals, 10 to 30 feet deep, by contrast continually showed accretion, with a total depth gain for the entire period of 8.16 feet. The channel areas have the same pattern as the marginal shoals — erosion to accretion — with a slight overall increase in depth of deposit, 0.29 foot. A similar general pattern was followed for Golden Gate sedimentation.

TABLE 21.—*Cumulative sediment volume change, North San Francisco Bay and Golden Gate*

Area	1855–95	1916	1948
	Million cubic yards	*Million cubic yards*	*Million cubic yards*
San Francisco Bay:			
Marginal shoals (0 to 10 feet) areas	−14.6	−5.6	−3.4
Intermediate shoals (10 to 30 feet)	65.6	96.1	161.1
Channel (deep water) areas	−53.5	−46.2	8.5
Totals	−2.5	44.3	166.2
	1855–73	1894	1956
Golden Gate	−8.1	−43.4	−10.8

South San Francisco Bay

Three general surveys, those of 1857, 1896,

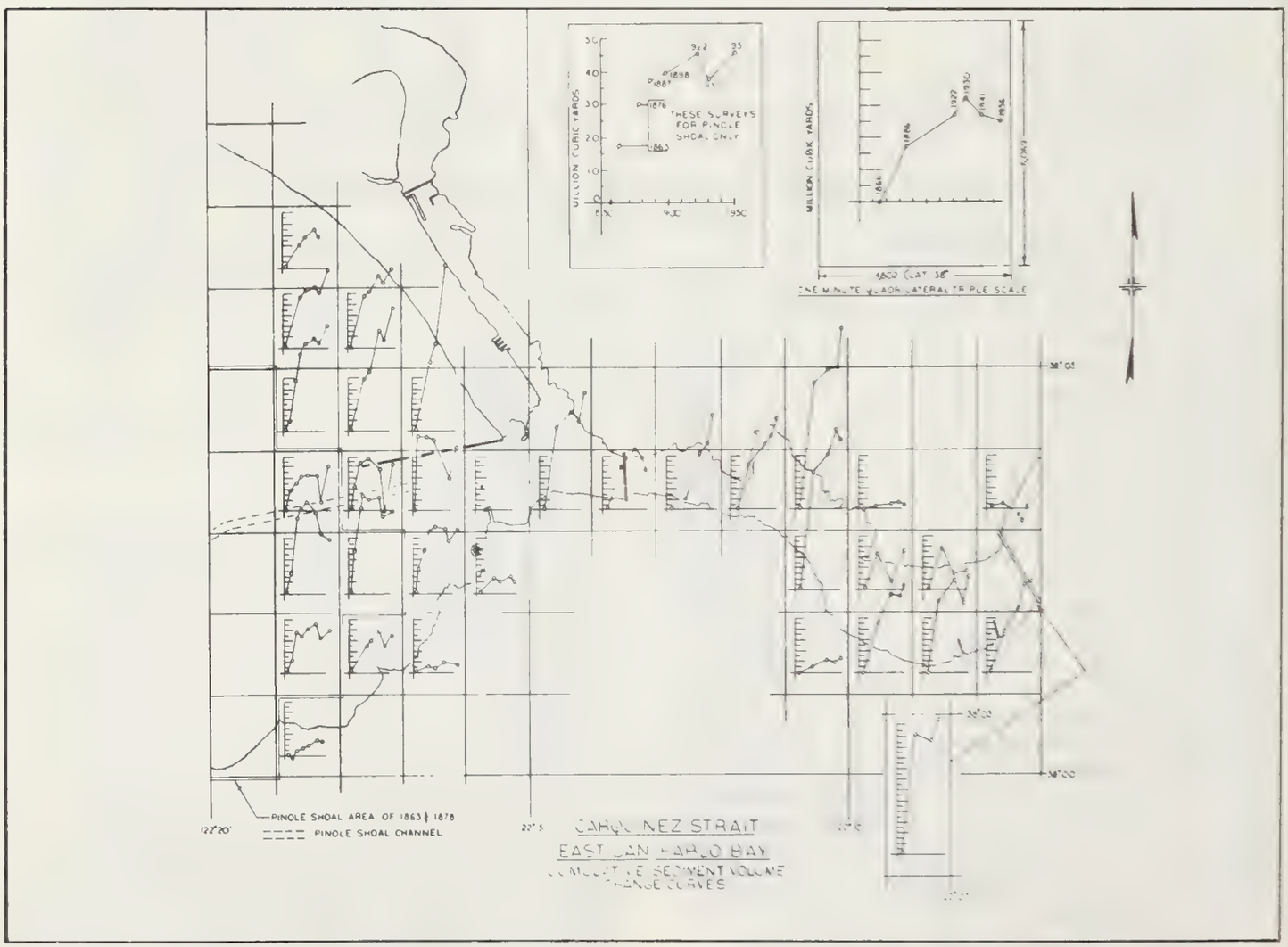


FIGURE 8. — Cumulative sediment volume change curves of Carquinez Strait, East San Pablo Bay.

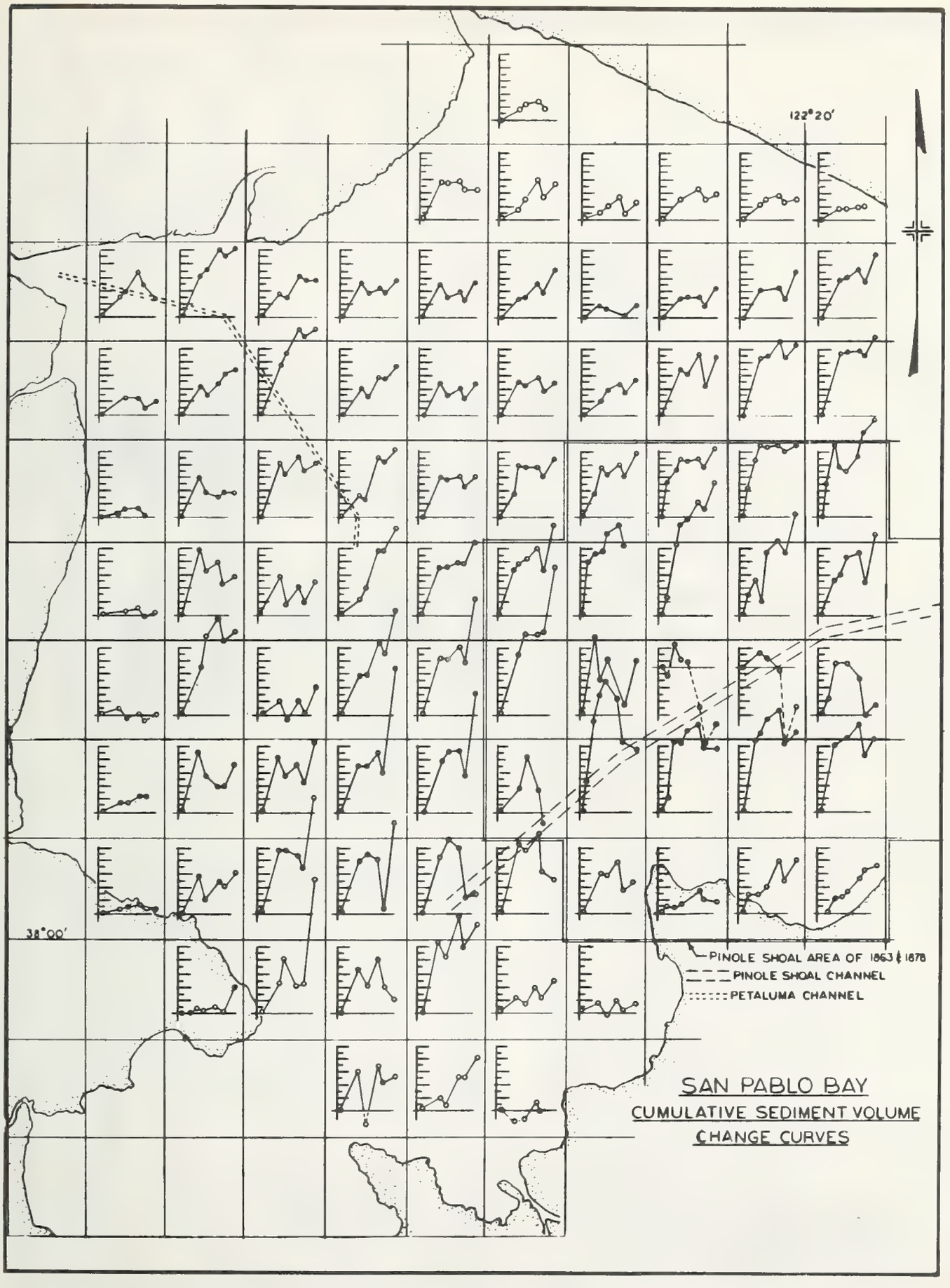


FIGURE 9. — Cumulative sediment volume change curves of San Pablo Bay.

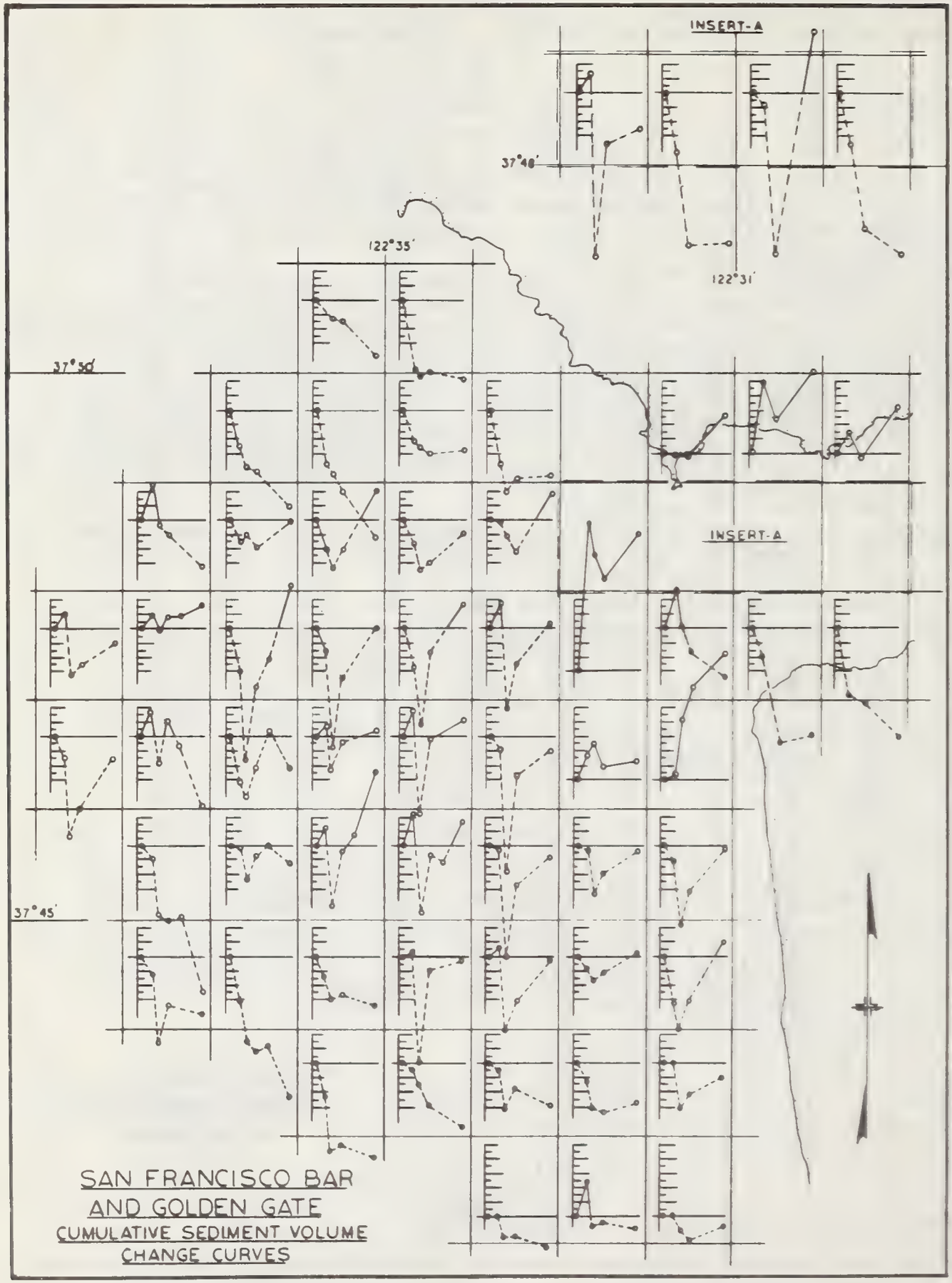


FIGURE 10. — Cumulative sediment volume change curves of San Francisco Bar and Golden Gate.

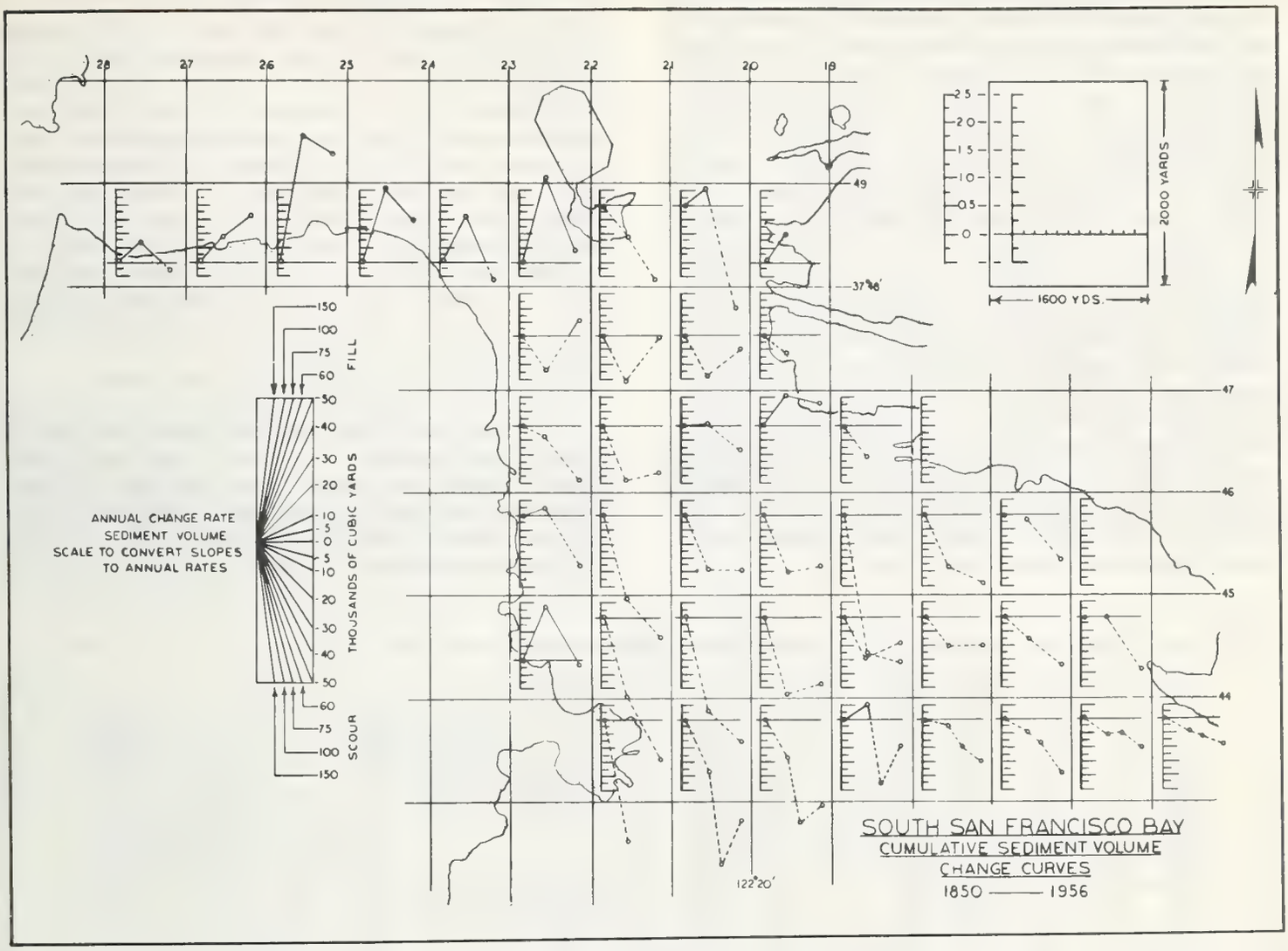


FIGURE 11. — Cumulative sediment volume change curves of South San Francisco Bay, 37° 49′ to 37° 43′ N.

and 1956, and a partial survey in 1931 are the basis for derivation of sediment volume changes in the South Bay, the largest bay in the system. The analysis is made over three zones: zone A covering 26 quadrilaterals from Treasure Island south to Hunters Point, lat. 37° 49′ to lat. 47° 44′; zone B covering 58 quadrilaterals from Hunters Point south to Point San Bruno, lat. 37° 44′ to lat. 37° 39′; and zone C covering 87 quadrilaterals from Point San Bruno south from lat. 37° 39′ to 37° 27′. Figures 11, 12, 13 and table 22 summarize the sediment volume changes.

Zone A, predominantly deep water, consistently shows scour over the past century, the change in mean depth being 1.7 feet. Scouring also occurs in zone B; light in the 40-year period 1855-96, a scour of 0.8 feet; continuing light to moderate, 0.9 foot, in the shorter period 1896-1922; and experiencing a light accretion, 0.15 foot, 1922-56. For zone C the general scour trend occurs after 1896, the period 1855-96 being one of silting (0.25 foot). The overall change for the South Bay is a scour of 173.5 million cubic yards, an erosion of 1 foot, over the period of 1855-1956.

San Francisco Bar

Between 1855 and 1956 the San Francisco Bar outside the Golden Gate has lost 85 million cubic yards (fig. 14). This figure is the result of careful recomputation of all available data. Comparative data for gains and losses in the outer and inner slopes and the crest are given in table 23. The general consensus of authorities studying the bar is that it is composed of sand eroded from the coast, swept into the Golden Gate by the floodtide and swept out on the ebbtide radially. Bay mud apparently forms no part of the bar. That part of fluvial sediment and disturbed bottom sediment of the Bay system that is carried out to the ocean is presumably swept past the bar.

A complete analysis of the San Francisco Bar

is not attempted, discussion being limited to consideration of the bar relative to sedimentation within the system. Between 1855 and 1873 there appears to be a general shrinkage of volume except in the deeper inner zone. This shrinkage, relatively slight — 30 million yards out of a present total volume above the 60-ft. depth of 874 million — was accelerated to 182 million cubic yards lost for the 11-year period 1873-84. From 1884 on, a reversal to accretion occurred — 77 million between 1884 and 1900 and a further 50 million between 1900 and 1956. A study of cross sections at time intervals between 1855 and 1957 also shows no marked movement, relatively, of the bulk of the bar, although a certain amount on internal displacement between zones has occurred. It is possible that the net shrinkage of 85 million yards is but a translation of volume to depths below the 60-ft. contour on the outer slope. This aspect is under study at the present time.

Discussion of Historic Sedimentation Pattern

Summarizing, the greater part (85.5 percent by weight) of the fluvial sediment load is delivered to the Delta. Direct inflows of sediment to the bays are 3.2 percent to Suisun Bay, 6.3 percent to San Pablo Bay, and 5.0 percent to San Francisco Bay. Upon entering the Bay system, the fluvial sediment loses the sand content and tends to flocculate as salinity increases. However, no simple pattern of deposition such as prevails for river regimens occurs within the bays and straits. Instead tide and wind effects and a long-term cyclic trend underlie the estuarial sedimentation processes.

Tidal Effects

The principal tidal effect is a net movement of sediment downbay, the major part being disturbed bottom sediment from the bay bottom, the minor part being the fluvial bedload. This is deduced from the absence of any dilution effect in the sediment load analysis of fluvial flow

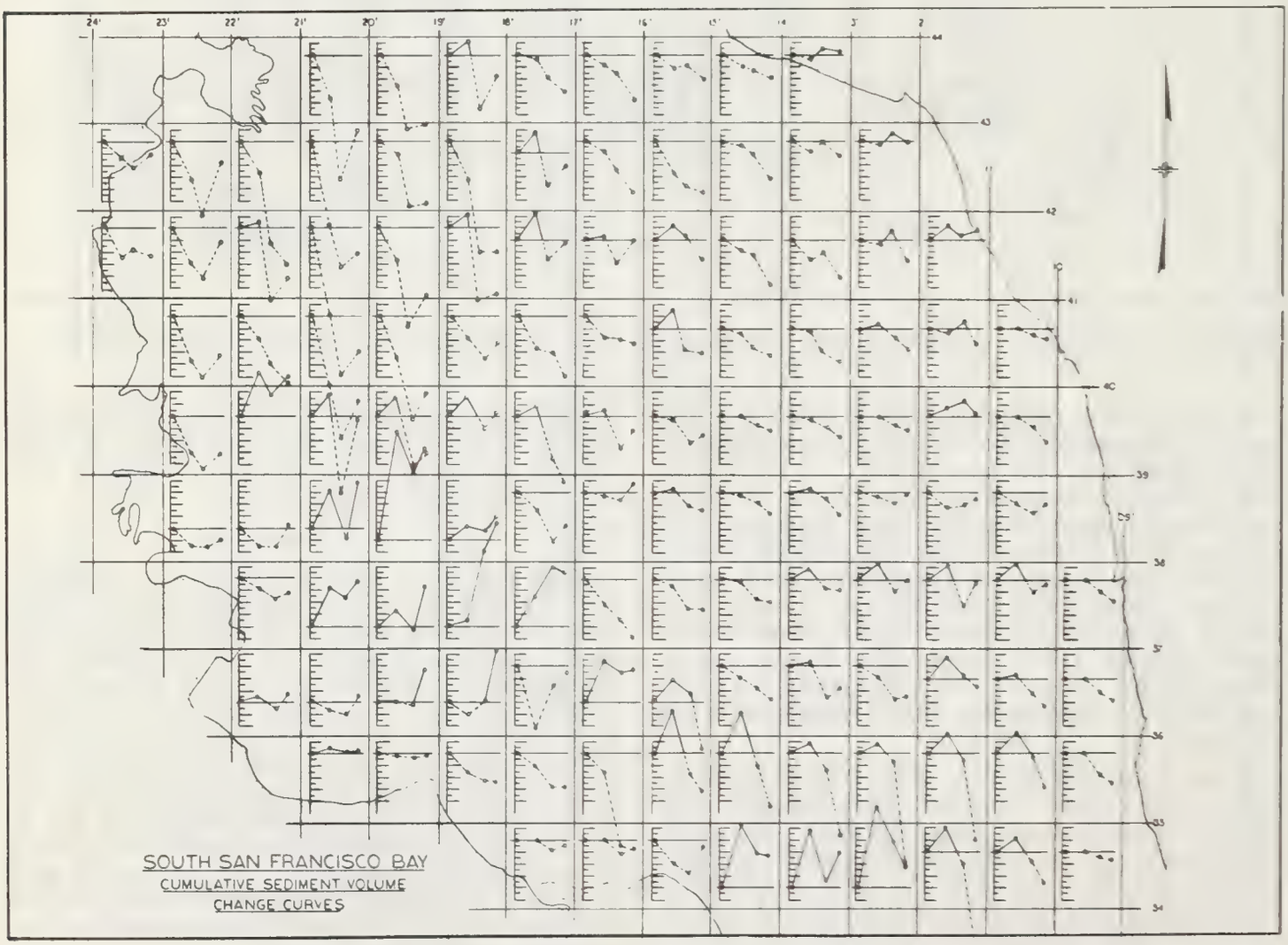


FIGURE 12. — Cumulative sediment volume change curves of South San Francisco Bay, 37° 43′ to 37° 35′ N.

sediment loadings and bay water sediment loadings. The effect of flocculation and reduction in grain-size dimensions results in lighter density bedload material. This is most marked in San Pablo Bay where flocculation is heaviest.

Movement downbay of both suspended load and bedload is the net result of the dominance of the ebb tidal competencies and capacities over those of the flood tide. This is illustrated in figure 15, which shows the high concentrations of sediment at peak flood and ebb and the dominance of the ebb over the flood.

Wind Effects

The effect of wind action on sedimentation over the large expanses of the several bays can be inferred from analysis of the deposition patterns by depth ranges. For shallow depths, wind wave turbulence would be pronounced. The stirring-up effect of the wave action would be augmented by the bottom return flow, opposite in direction to the wind-induced surface flow. Progressively as the fetch increased, the turbulence and volume of sediment in suspension would increase, reducing the potential for deposition. For Suisun Bay the prevailing winds are from the northwest with Honker Bay the farthest east, Grizzly Bay next, and Suisun Slough farthest west. Hence, for the same periods, siltation over the shallow areas should be greatest in the most westerly parts and lightest in the most easterly.

This assumes not too marked a variation in salinity over the area. From table 18 the cumulative sediment volume changes show deposition occurring in descending order of volume for the shallow areas from highs in the western to lows in the eastern parts of the Bay, the least deposition occurring in Honker Bay, the farthest east. For the period 1867-86 this effect is pronounced, the ratio of deposition per unit area of shallows being approximately 4:1 from west (Suisun Slough and West Bay) to east (Honker Bay). The effect is masked in the

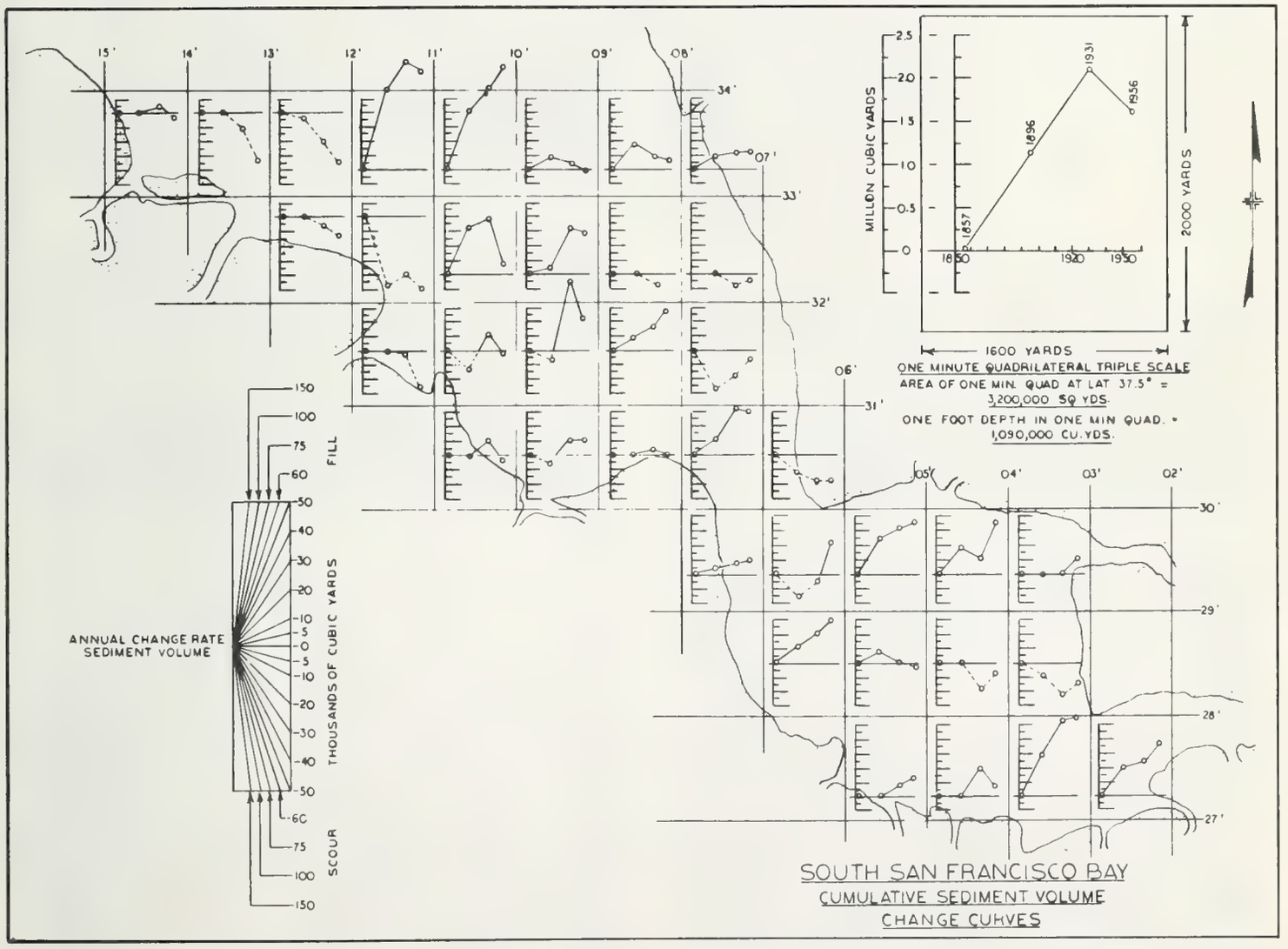


FIGURE 13. — Cumulative sediment volume change curves of South San Francisco Bay, 37° 35′ to 37° 27′ N.

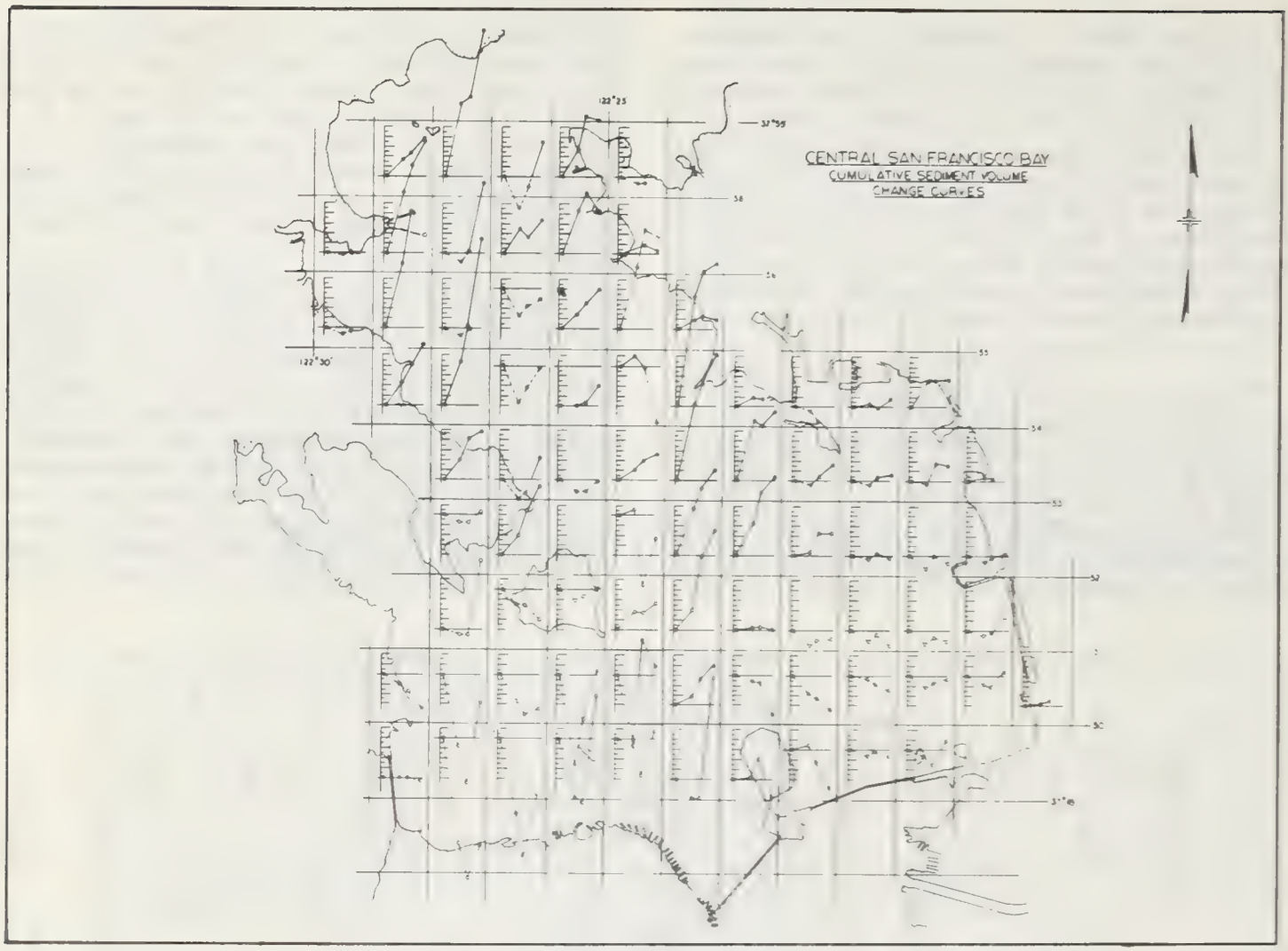


FIGURE 14. — Cumulative sediment volume change curves of Central San Francisco Bay.

drought period 1921–30 by changed flocculation conditions when the saline-fresh water interface apparently moved from San Pablo Bay into Suisun Bay.

When the shallow areas are grouped for San Pablo Bay, the rate of the deposition (see table 20) for the marginal shoals (0 to 4 feet) is 2.2 times that for the intermediate shoals (4 to 20 feet). This is not so apparent by inspection of figures 8 and 9, due to the orientation of the shallows and also the masking of the wind-induced turbulence effect by the fluctuations of salinity that occur across the Bay as the saline interface moves with seasonal fluctuations of fresh water inflows. The construction of dike 12 from the southern end of Mare Island westerly from 1,000 feet in 1908 to a total length of 12,800 feet between 1937 and 1951 materially altered the deposition pattern in the vicinity of Mare Island. Data on changes in bottom elevations along the dike are given in table 24. The accentuated accretion after 1896 can be attributed to the impounding effect of the dike.

The detailed aspects of shoaling in Mare Island Strait and the South and North Bays are presented by W. J. Homan and E. A. Schultz (*9*) and are here discussed in general terms only.

Wind wave effects on sedimentation in North San Francisco Bay, the central Bay of the system, and in South Bay are masked by the complex interactions of tidal currents and phases. The deposition rates of the marginal shoals, 0 to 10 feet deep, in the North Bay are markedly less (table 21) than the rates for the intermediate shoals, 10 to 30 feet deep. However, the Berkeley flats that constitute most of the marginal shoals are to the extreme east of the Bay where the fetch is greatest from the west. Both these flats and the Southampton Shoal are probably more affected by the tidal complex. Similarly, for the South Bay no adequate explanation is yet available for the formation of San Bruno Shoal and the silting in the vicinity of Alameda

TABLE 22.—*Cumulative volume and mean depth changes, South San Francisco Bay, 1855–1956*

Zone volume and depth	1855–96	1922	1931	1956
A, Treasure Island to Hunters Pt.:				
Volume, million cu. yds.	−29.3	.....	.....	−47.7
Depth, ft.	− 1.10	.....	.....	− 1.70
B, Hunters Pt. to Pt. San Bruno:				
Volume, million cu. yds.	−51.5	−106.5	.....	−95.9
Depth, ft.	− 0.80	− 1.70	.....	− 1.55
C, Pt. San Bruno south:				
Volume, million cu. yds.	+25.2	.....	−14.5	−29.9
Depth, ft.	+ 0.25	.....	− 0.15	− 0.30
Total:				
Volume, million cu. yds.	−55.6	.....	.....	−173.5
Depth, ft.	− 0.30	.....	.....	− 1.00

Island. Figures 16, 17, 18, and 19, showing the shoal areas and typical cross section of the major bays and the wind roses at strategic points, indicate the extensive areas of the system subject to wind action.

Interactioning Within the System

An interchange of water volumes, concentrated at the turn of the tides, occurs between the South Bay and the north bays with consequent interchange of sediment volumes. Much more coordinated prototype data coupled with model tests are needed to prove the workings of this sediment interchange. Figure 20 and table 25 present data of interest but are inadequate to form a base for firm conclusions. Over the 101-year period, the South Bay has undergone a net scour of 173.5 million cubic yards, while the North Bay has experienced a net gain of 166.1 million yards, a virtual balance.

Further indications of interactioning within the system are the relations between sedimentation in Suisun and San Pablo Bays. In this instance the coincidence of survey dates with the dry period 1922-30 is advantageous. Between 1886 and 1941 Suisun Bay was subject to scour except for the dry period 1922-30. San Pablo Bay between 1886 and 1951 experienced accretion except during 1922-30, when scour occurred the reverse to Suisun Bay. There is little doubt that normally the heavy sedimentation in San Pablo Bay reflects heavy flocculation and that during 1922-30 the migration into Suisun Bay of the saline–fresh water interface resulted in the reversed pattern for both bays.

TABLE 24.—*Historical bottom elevations (MLLW) along north side of dike 12, certain years*

Year	Distance along dike crest from east end				
	200 feet	2,000 feet	7,500 feet	8,500	200 ft. beyond end of dike
	Feet	*Feet*	*Feet*	*Feet*	*Feet*
1856	−7	−33	−27	−27	−19
1878	.....	.....	.....	.....	−20
1896	−4	−25	−27	−26	−23
1922	.....	.....	.....	−24	−20
1951	(1)	+2	+2	0	−8

[1] Shoreline.

Environmental Aspects

From the biological environmental viewpoint, the most important character of the Bay system is the prevailing shallowness of the bays, with a few such exceptions as Golden Gate, Carquinez Strait, and Raccoon Strait. Depths 15 feet and less below mean sea level occupy 70 percent of the total water area, and at —20 feet the water would be but 21 percent of the total at mean sea level. This condition can be highly productive

TABLE 23.—*San Francisco Bar and adjacent offshore area sediment volume change*

Area, volume, and depth	1855–73	1873–84	1873–1900	1884–1900	1900–1956	Cumulative sediment volume change
Inner slope above 60 ft.—11 quads.:						
Volume, million cubic yards	−4.4	−61.5	−17.6	+43.9	+29.0	+7.0
Depth, feet	−0.36	−5.16	−1.47	+3.69	+2.44	.....
Crest inside 36-foot contour—11 quads.:						
Volume, million cubic yards	−18.2	−18.2	−10.3	+7.9	+11.0	−17.4
Depth, feet	−1.53	−1.53	− .87	+0.66	+0.93	.....
Outer slope above 60 ft.—18 quads.:						
Volume, million cubic yards	−20.4	−63.6	−53.1	+10.5	−8.3	−81.8
Depth, feet	−1.04	−3.26	−2.73	+0.54	−0.43	.....
Miscellaneous—below 60 ft.—10 quads.:						
Volume, million cubic yards	+13.1	−38.5	−23.6	+14.9	+17.7	+7.2
Depth, feet	+1.21	−3.55	−2.18	+1.38	+1.63	.....
Entire 50 quads.:						
Volume, million cubic yards	−29.9	−181.8	−104.6	+77.2	+49.5	−85.0
Depth, feet	−0.54	−3.36	−1.93	+1.43	+0.91	−1.5

TABLE 25.—*Adjusted historical sedimentation, San Francisco Bay System, 1855–1956*

Area	Cycle						Net sedimentation, 101 years[1]	
	Accretion			Scour			Rate	Total volume
	Years	Rate	Volume	Years	Rate	Volume		
		Feet	*10^6 cu. yd.*		*Feet*	*10^6 cu. yd.*		
Suisun Bay, except Grizzly Bay	1855– 86 (31)	+ 1.85	+ 57.4					
Do				1886–1922 (36)	− 0.95	−34.2		
Do	1922– 30 (8)	+ 1.24	+ 9.9					
Do				1930– 56 (26)	− 1.57	−40.1		
Grizzly Bay	1855– 86 (31)	+ 1.68	+ 52.1					
Do				1886–1956 (70)	− .02	− 1.4		
Subtotal			+119.4			−75.7	+0.43	+ 43.7
Carquinez Strait	1855– 86 (31)	+ 2.41	+ 74.7					
Do	1886–1922 (36)	+ .30	+ 11.0					
Do	1922– 56 (34)	+ .62	21.1					
Subtotal			+106.8				+1.06	+106.8
San Pablo Bay	1855– 87 (32)	+10.45	+334.4					
Do	1887– 99 (12)	+ 41	+ 4.9					
Do	1899–1922 (23)	+ 2.66	+ 61.1					
Do				1922– 30 (8)	−11.2	−89.6		
Do	1930– 56 (26)	+ 5.35	+139.1					
Subtotal			+539.5			−89.6	+4.45	+449.9
North San Francisco Bay				1855– 95 (40)	− .063	− 2.5		
Do	1895–1916 (21)	+ 2.23	+ 46.9					
Do	1916– 56 (40)	+ 3.8	+152.0					
Subtotal			+198.9			− 2.5	+1.94	+196.4
Golden Gate				1855– 73 (18)	− 0.45	− 8.1		
Do				1873– 94 (21)	− 1.68	− 35.3		
Do	1894–1956 (62)	+ 0.53	+ 32.6					
Subtotal			+ 32.6			— 43.4	−0.11	− 10.8
South San Francisco Bay				1855– 96 (41)	− 1.39	− 55.6		
Do				1896–1956 (60)	− 1.965	−117.9		
Subtotal						−173.5	−1.72	−173.5
Total, San Francisco Bay	1855–1956	+ 9.87	+997.2	1855–1956	− 3.81	−384.7	+6.06	+612.5
San Francisco Bar				1855– 73 (18)	− 1.66	− 29.9		
Do				1873– 84 (11)	−16.53	−181.8		
Do	1884–1900 (16)	+ 4.83	+ 77.2					
Do	1900– 56 (56)	+ .88	+ 49.5					
Subtotal			+126.7			−211.7	− .84	− 85.0
Total, Bay plus San Francisco Bar	1855–1956	11.13	+1,123.9	1855–1956	− 5.91	−596.4	+5.22	+527.5

[1] Adjustment to 101 years made by extending back to 1855 or forward to 1956 for areas lacking data from 1855 or to 1956 at the rates of the nearest cycle in time.

for aquatic life, emergent and nonemergent vegetation, and shore life. Normally estuarine conditions inhibit great population concentrations of marine life, but for the Bay system large areas of relative quiescence are conducive to bottom life such as oysters. The productive oyster bed area for the system has been estimated at 170,000 acres of bay floor with a potential yield up to 400 million pounds a year (*6*). Also the mud flats and shallows, in addition to oysters and clams, support shore bird life.

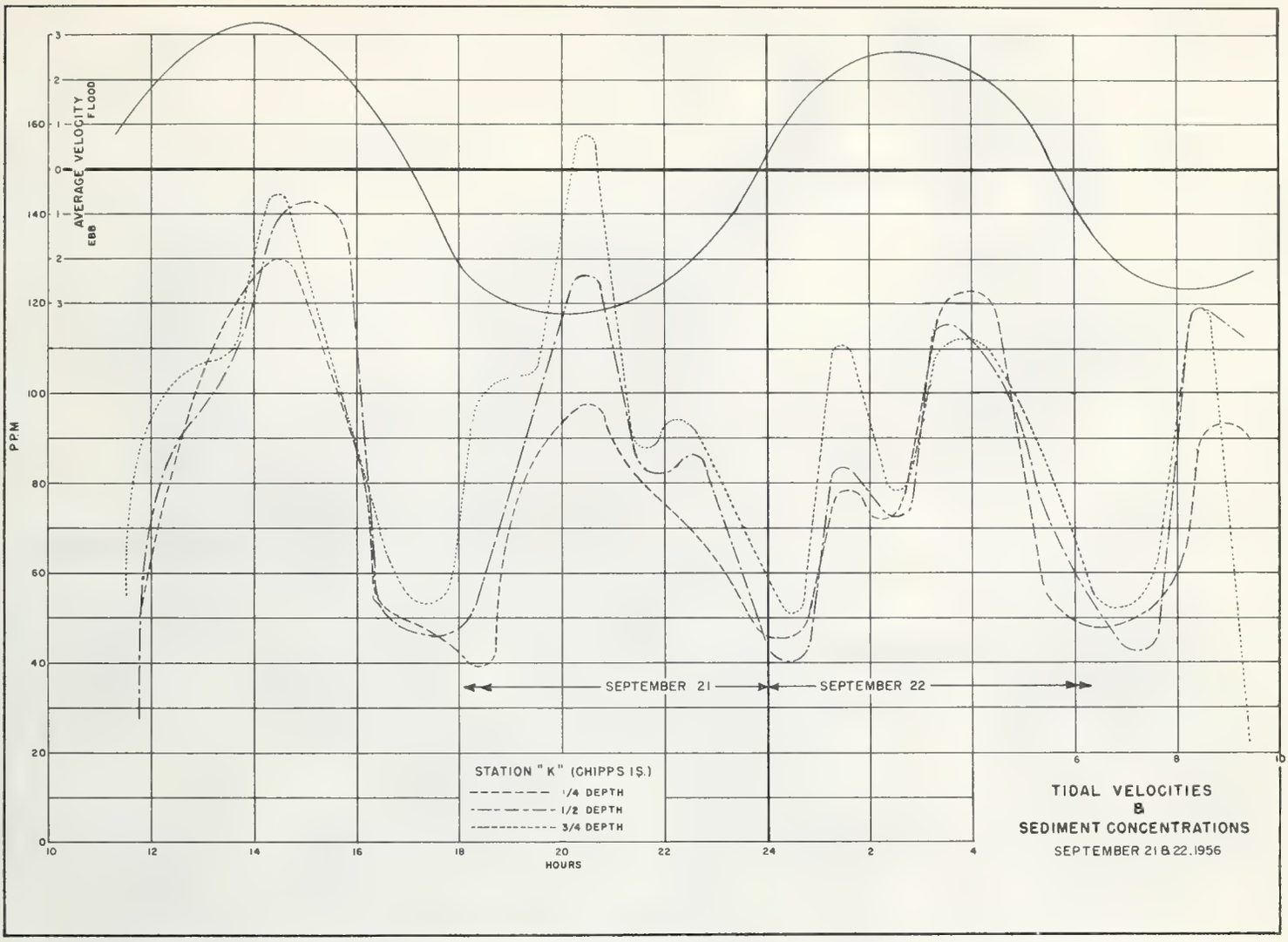


FIGURE 15. — Tidal velocities, phases, and sediment concentrations near Chipp Island, September 21 and 22, 1956.

Sedimentation by affecting turbidity and being the chief carrier of nutrient material for the estuarine biota is an important basic factor in the biotic environment. Parallel factors of equal rank are the saline and temperature gradients throughout the system (*4*, *5*).

Hence, any major displacement of sediment volumes between bays resulting from cyclic changes such as the wet-drought cycle or artificial redeployment of basic areal characteristics would have major effects on the environmental conditions sustaining the biota. Essentially such major displacement would come from altered salinity, thereby changing the flocculation pattern, altered tidal current strengths, direction and phasing affecting deposition and turbidity, altered bottom configurations affecting depths, and corresponding habitats.

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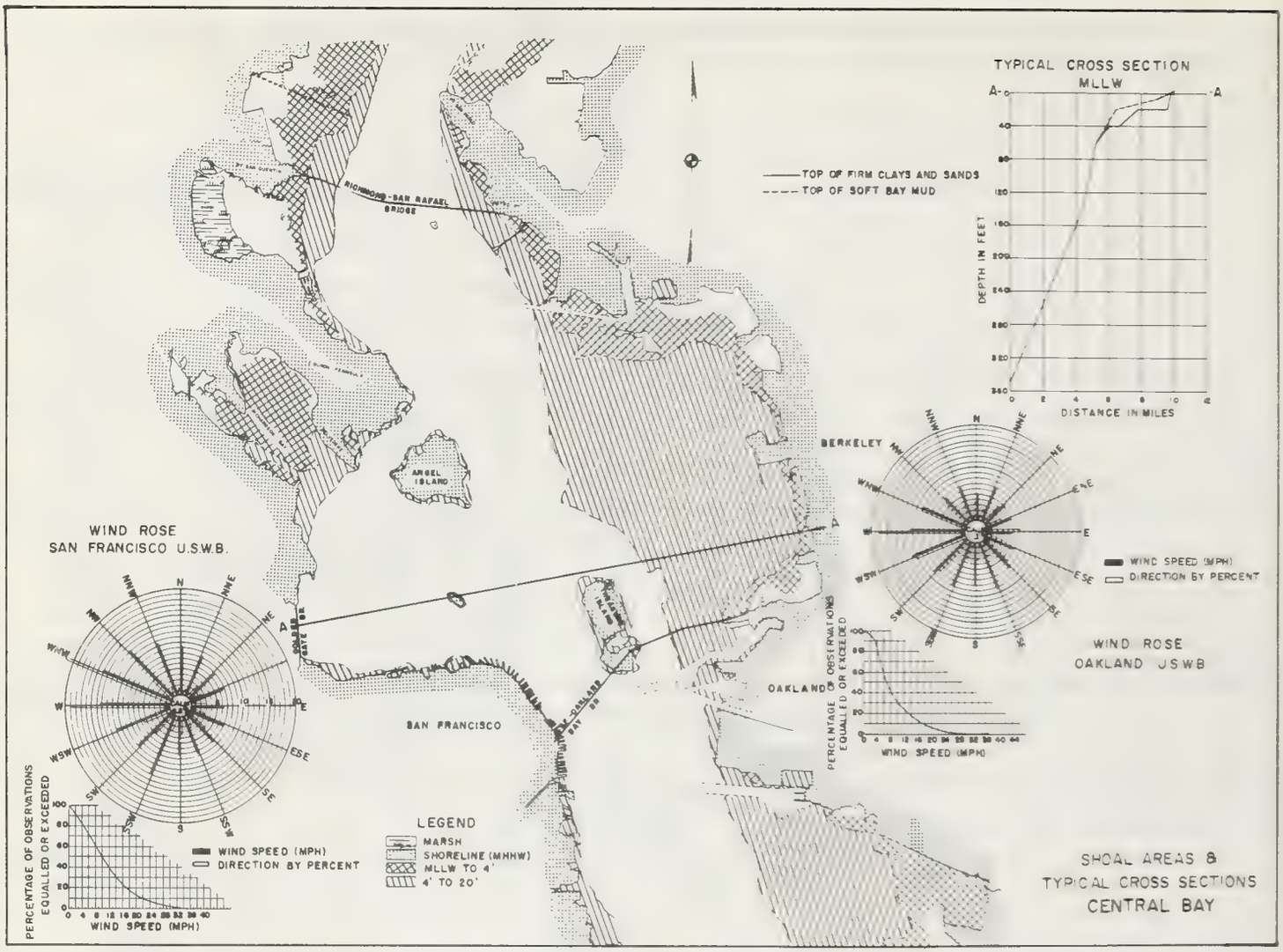

FIGURE 16. — Shoal areas, wind roses, and typical cross section of Central San Francisco Bay.

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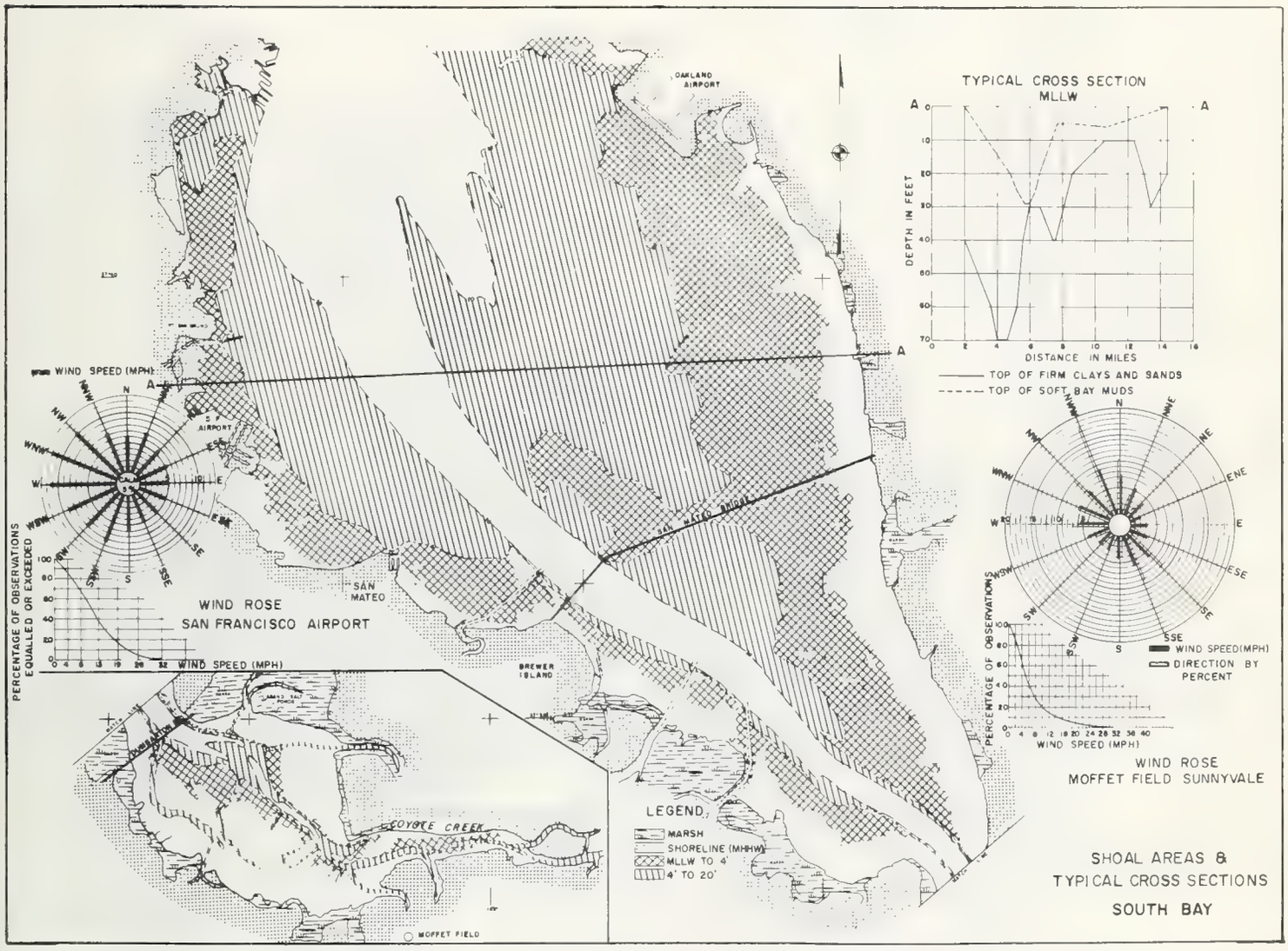


FIGURE 17. — Shoal areas, wind roses, and typical cross section of South San Francisco Bay.

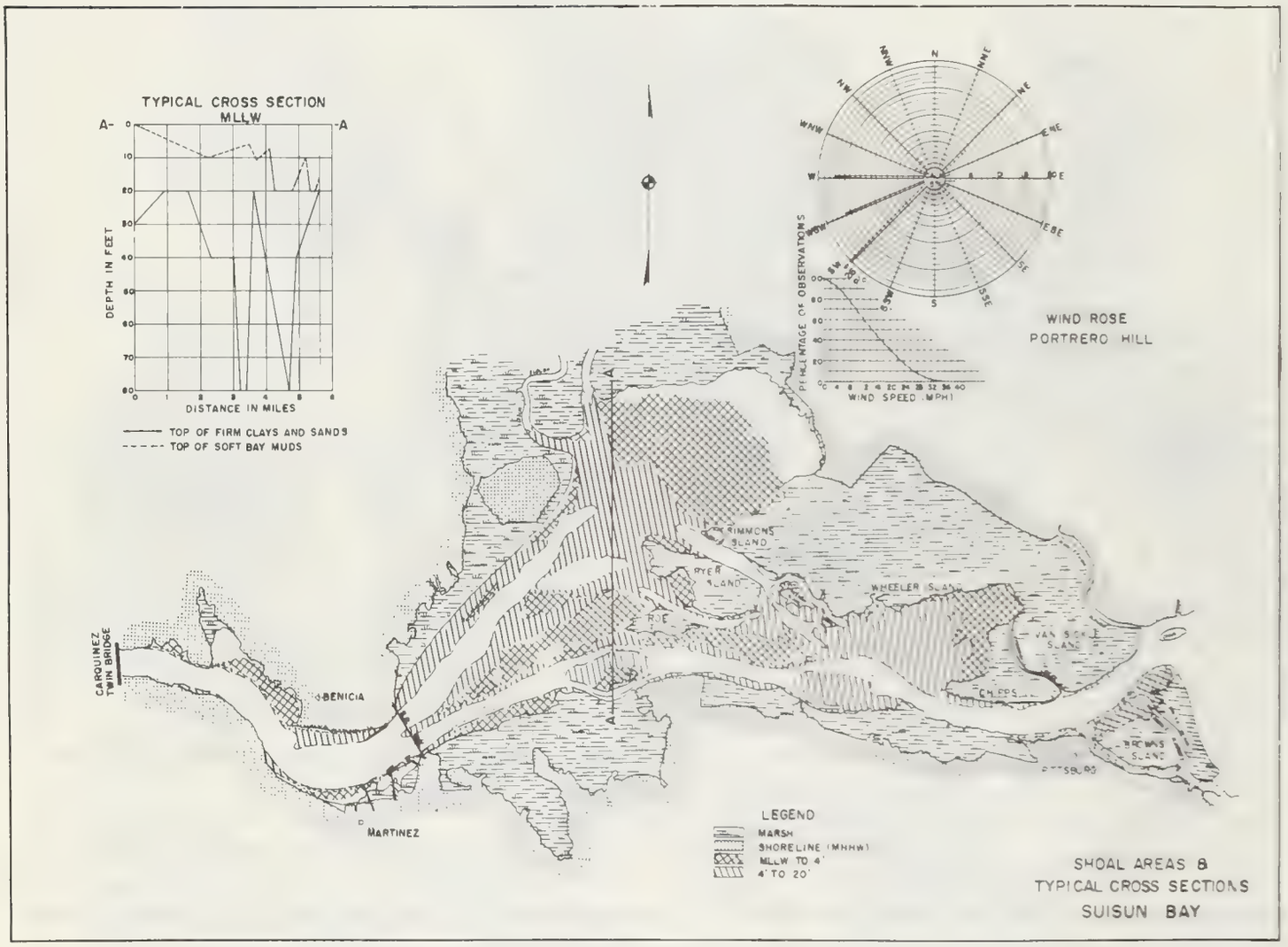


FIGURE 18. — Shoal areas, wind roses, and typical cross section of Suisan Bay.

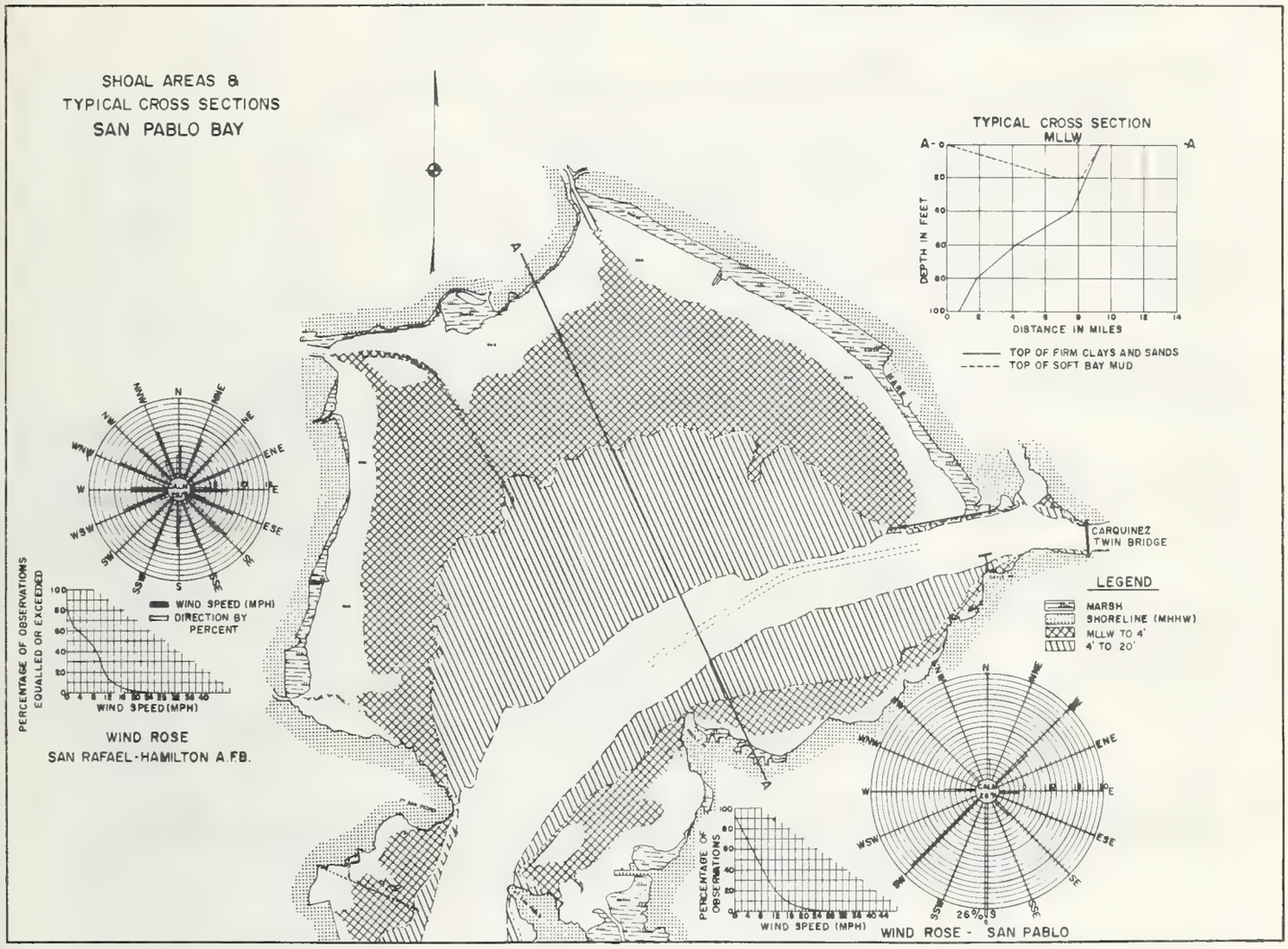


FIGURE 19. — Shoal areas, wind roses, and typical cross section of San Pablo Bay.

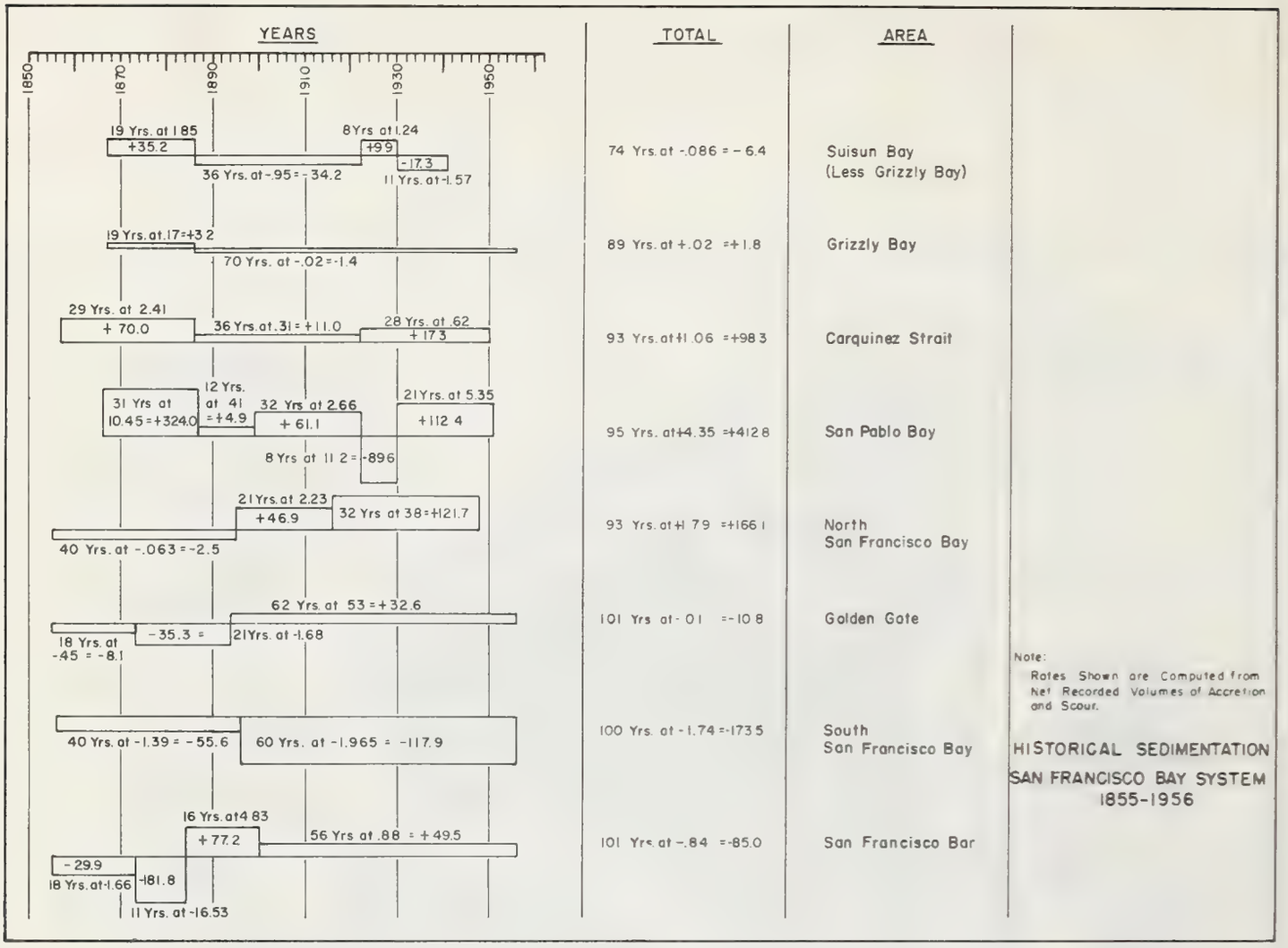


FIGURE 20. — History of sedimentation in San Francisco Bay system, 1855-1956.

MODEL TESTS OF SHOALING AND OF DREDGE SPOIL DISPOSAL IN SAN FRANCISCO BAY

[Paper No. 71]

By W. J. HOMAN, *supervisory civil engineer (general)*, and E. A. SCHULTZ, *supervisory hydraulic engineer (research), U.S. Army Engineer District, San Francisco*

Abstract

San Francisco Bay has an area of 486 square miles. The tributary area is 62,920 square miles. Much of the Bay is very shallow: 70 percent is less than 20 feet deep. The average annual silt deposition rate has been about 6,000,000 cubic yards. Estimated future sediment inflow is 5,840,000 tons a year.

There are 17 authorized navigation projects in the Bay system below Carquinez Strait. The Federal Government spends about $2,270,000 annually for maintenance dredging. Dredge spoils are dumped in deep sections of the Bay.

Model tests indicate approximately 30 percent of the material dredged from the Oakland-Alameda area is carried to sea and about 4 percent returns to navigation channels; and that dredge spoils can be dumped more than 1 mile closer to the point of origin without detrimental results. Model tests of Mare Island Strait indicate similar possibilities. Material originally deposited in shallow areas may be resuspended by wave action and returned to navigation channels.

Model tests of remedial works at the Alameda Naval Air Station indicate possible sediment reductions of as much as 69 percent.

Savings in maintenance dredging as a result of model studies are expected to far exceed the cost of such studies.

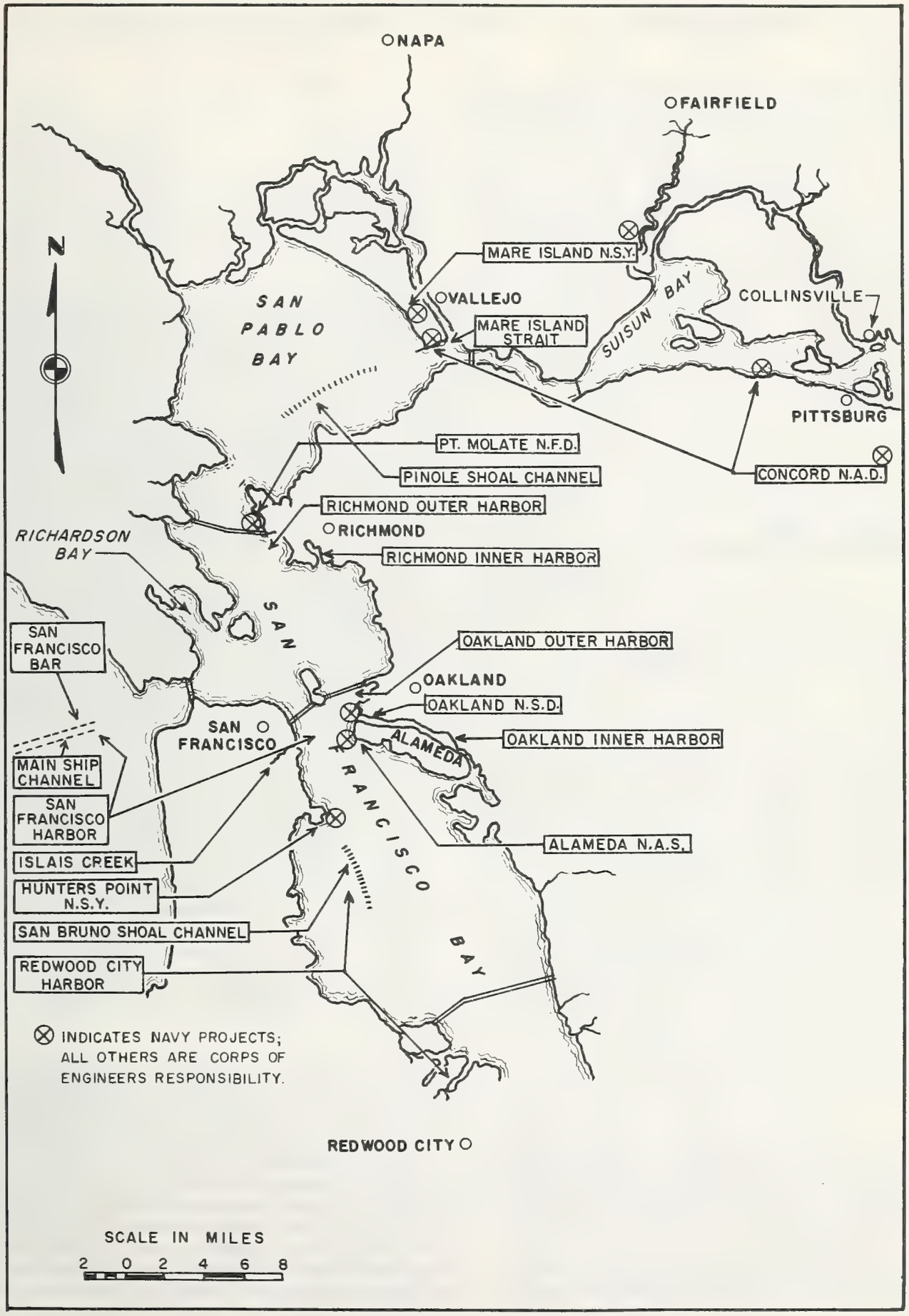


FIGURE 1. — San Francisco Bay.

Introduction

The Bay

San Francisco Bay, shown in figure 1, contains one of the largest and most important port complexes on the west coast of the United States, and is also the discharge point for the largest drainage basin in the State of California — the Sacramento-San Joaquin River system, which drains the great Central Valley of California. The active area contributing to the discharge into San Francisco Bay contains 48,883 square miles. There is an additional area of 14,037 square miles that is normally landlocked, but which, during extreme wet cycles, might contribute to the discharge into the Bay. The total tributary area is thus 62,920 square miles, or about 40 percent of the total area of the State. This area is shown in table 1.

TABLE 1.—*Area tributary to San Francisco Bay*

Area	Drainage size
	Square miles
Sacramento River Basin	26,322
San Joaquin River Basin	19,096
Small streams discharging directly into Bay	3,465
Total	48,883
Tulare Lake Basin (south of San Joaquin Basin)	13,625
Goose Lake Basin (north of Sacramento Basin)	412
Grand total	62,920

Although the channel through the Golden Gate has a maximum depth of about 385 feet, San Francisco Bay, as a whole, is relatively shallow and bordered by extensive areas of "tidelands" that are exposed at low tide. There is also a crescent-shaped bar about 5 miles seaward from the Golden Gate. The natural water depth at the deepest point over the crest of this bar is about 34 feet at mean lower low water. The area inside the Golden Gate below the junction of the Sacramento and San Joaquin Rivers at Collinsville, which excludes the Sacramento-San Joaquin Delta with an area of about 34,000 acres at mean sea level, at various elevations, and percentage of area at mean sea level at these same elevations is shown in table 2.

Sedimentation

A recent estimate, by the U. S. Geological Survey, of silt expected to be discharged into the Bay system under future conditions is 5,840,000 tons a year. About 1,940,000 tons (1,600,000 cu. yd. at 90 lb. per cu. ft.) is deposited in the Sacramento-San Joaquin delta, leaving a balance of 3,900,000 tons entering the Bay proper. This is made up chiefly of fine particles in a collodial or semicolloidal suspension. Much

TABLE 2.—*Area at various elevations in San Francisco Bay, 1962*

Elevation	Area	Area	Area at mean sea level
	Acres	*Square miles*	*Percent*
Mean sea level	278,600	435	100
Mean higher high water	294,400	460	106
Mean lower low water	253,400	396	91
− 5 feet mean sea level	199,000	311	71
−10 feet mean sea level	138,500	216	50
−15 feet mean sea level	101,700	159	37
−20 feet mean sea level	87,600	137	32

of this material is precipitated by flocculation on contact with salt water and deposited in the Bay. Sediment originally deposited in shallow areas may be resuspended by wind and wave action and a part of material thus resuspended may be deposited in navigation channels or carried to sea through the Golden Gate. The total volume of shoals formed in the Bay is dependent on the inplace density developed by the sediment. Available data indicate average inplace densities ranging from 30 to 40 pounds per cubic foot, which is equal to a total sediment inflow between 7.2 and 9.6 million cubic yards a year.

A study of the historical shoaling pattern of San Francisco Bay shows somewhat extreme variations. These studies also show nonuniform distribution throughout the Bay; in fact, there have been periods of degradation in almost all sections of the Bay and there appears to be a long time trend of degradation in the South Bay. Studies of the shoaling pattern and trend for the 101-year period from 1855-1956 indicate an average annual deposition of 6,060,000 cubic yards for the Bay as a whole. These studies are based on a comparison of water depths as obtained from navigation charts and surveys by the U.S. Coast and Geodetic Survey. There is some question as to the accuracy of some of this work, due in part to changes in both vertical and horizontal controls. However, findings are believed to be indicative of long time trend. Findings for the 101-year period are shown in table 3.

Tides

Tides in the San Francisco Bay system are of the mixed type; there being two high tides and two low tides during each lunar day of 24 hours and 50 minutes. There is a marked and constantly changing difference in these tides. There is likewise a difference in the type of tide in the southern section of the Bay and in the central and northern section and up the Sacramento and San Joaquin River channels. Water surface elevations in the Sacramento River are affected by tides as far upstream as the vicinity of Knights Landing, some 20 miles above Sacra-

TABLE 3.—*Adjusted historical sedimentation, San Francisco Bay system, 1855–1956*

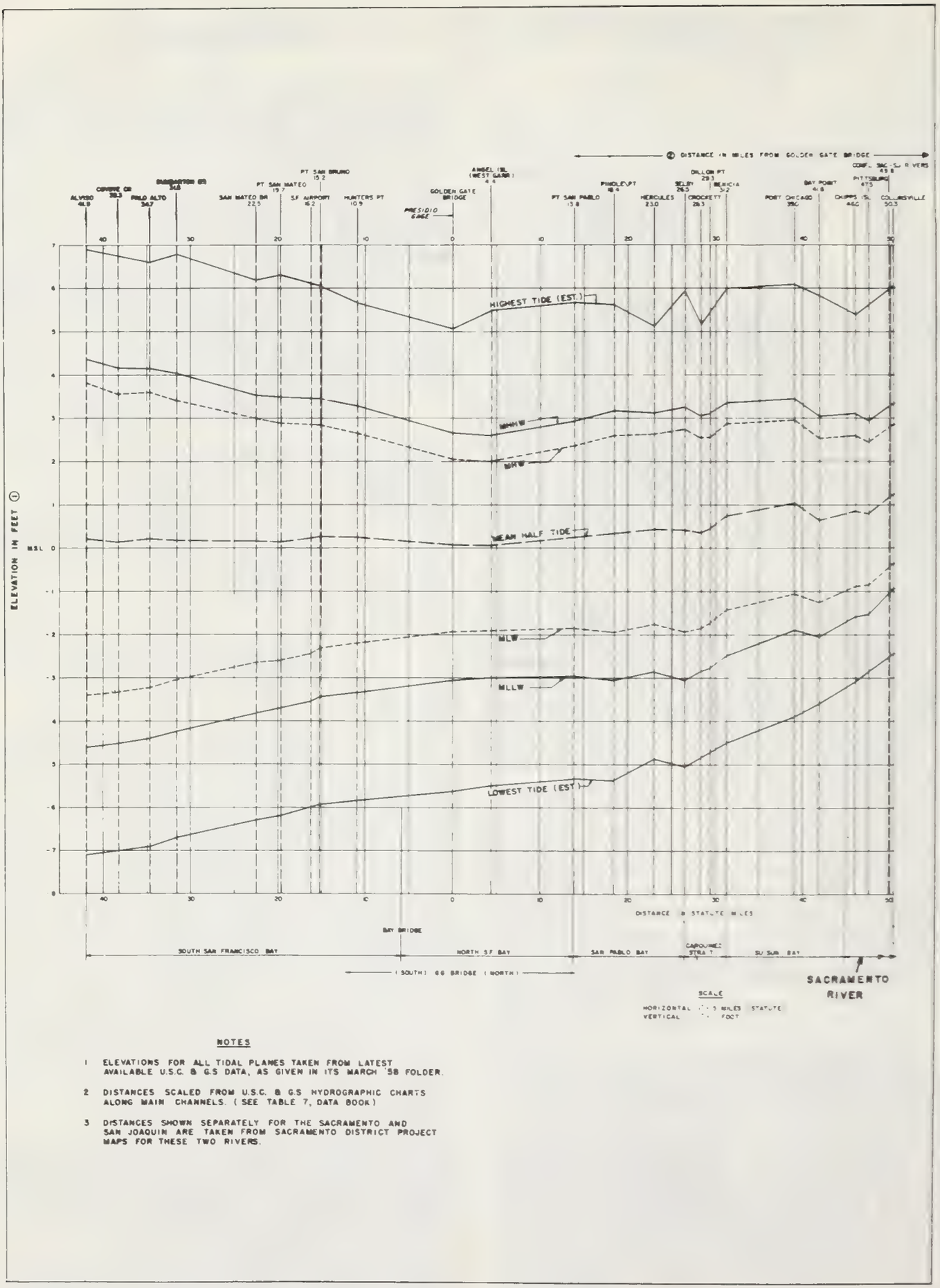
Area	Cycle						Net sedimentation, 101 years[1]	
	Accretion			Scour			Rate	Total volume
	Years	Rate	Volume	Years	Rate	Volume		
		Feet	*10^6 cu. yd.*		*Feet*	*10^6 cu. yd.*	*Feet*	*10^6 cu. yd.*
Suisun Bay, except Grizzly Bay	1855– 86 (31)	+ 1.85	+ 57.4					
Do				1886– 22 (36)	− 0.95	−34.2		
Do	1922– 30 (8)	+ 1.24	+ 9.9					
Do				1930– 56 (26)	− 1.57	−40.1		
Grizzly Bay	1855– 86 (31)	+ 1.68	+ 52.1					
Do				1886– 56 (70)	− .02	− 1.4		
Subtotal			+119.4			−75.7	0.43	+ 43.7
Carquinez Strait	1855– 86 (31)	+ 2.41	+ 74.7					
Do	1886– 22 (36)	+ .30	+ 11.0					
Do	1922– 56 (34)	+ .62	21.1					
Subtotal			+106.8				+1.06	+106.8
San Pablo Bay	1855– 87 (32)	+10.45	+334.4					
Do	1887– 99 (12)	+ .41	+ 4.9					
Do	1899– 22 (23)	+ 2.66	+ 61.1					
Do				1922– 30 (8)	−11.2	−89.6		
Do	1930– 56 (26)	+ 5.35	+139.1					
Subtotal			+539.5			−89.6	+4.45	+449.9
North San Francisco Bay				1855– 95 (40)	−0.063	− 2.5		
Do	1895– 16 (21)	+2.23	+ 46.9					
Do	1916– 56 (40)	+ 3.8	+152.0					
Subtotal			+198.9			− 2.5	+1.94	+196.4
Golden Gate				1855– 73 (18)	−0.45	− 8.1		
Do				1873– 94 (21)	−1.68	− 35.2		
Do	1894– 56 (62)	+ .53	+ 32.6					
Subtotal			+ 32.6			− 43.4	− .11	− 10.8
South San Francisco Bay				1855– 96 (41)	−1.39	− 55.6		
Do				1896– 56 (60)	−1.965	−117.9		
Subtotal						−173.5	−1.72	−173.5
Total, entire San Francisco Bay	1855– 56	+ 9.87	+997.2	1855– 56	− 3.81	−384.7	+6.06	+612.5
San Francisco Bar				1855– 73 (18)	−1.66	− 29.9		
Do				1873– 84 (11)	−16.53	−181.8		
Do	1884– 00 (16)	+ 4.83	+ 77.2					
Do	1900– 56 (56)	+ .88	+49.5					
Subtotal			+126.7			−211.7	− .84	− 85.0
Total, Bay plus Bar	1855–1956	11.13	+1,123.9	1855–1956	−5.91	−596.4	+5.22	+527.5

[1] Adjustment to 101 years made by extending back to 1855 or forward to 1956 for area lacking data from 1855 or to 1956 at the rates of the nearest cycle in time.

mento; and on the San Joaquin River to a few miles upstream from the Mossdale Bridge where Highway 50 crosses the river. Tides in the South Bay are predominantly of the standing wave type, which results in very poor circulation in the southern section of the South Bay. Those in the central and northern section are of the progressive wave type. Tidal heights are shown on the tidal reference plane chart figures 2 and 3. Tidal elevations referred to mean sea level at the Golden Gate and ranges at various key points are shown in table 4.

Maintenance Dredging

The Federal Government spends about $2,270,000 annually for maintenance dredging in



NOTES

1 ELEVATIONS FOR ALL TIDAL PLANES TAKEN FROM LATEST AVAILABLE U.S.C. & G.S DATA, AS GIVEN IN ITS MARCH '58 FOLDER.

2 DISTANCES SCALED FROM U.S.C. & G.S HYDROGRAPHIC CHARTS ALONG MAIN CHANNELS. (SEE TABLE 7, DATA BOOK)

3 DISTANCES SHOWN SEPARATELY FOR THE SACRAMENTO AND SAN JOAQUIN ARE TAKEN FROM SACRAMENTO DISTRICT PROJECT MAPS FOR THESE TWO RIVERS.

FIGURE 2. — Tidal reference plane chart, San Francisco Bay.

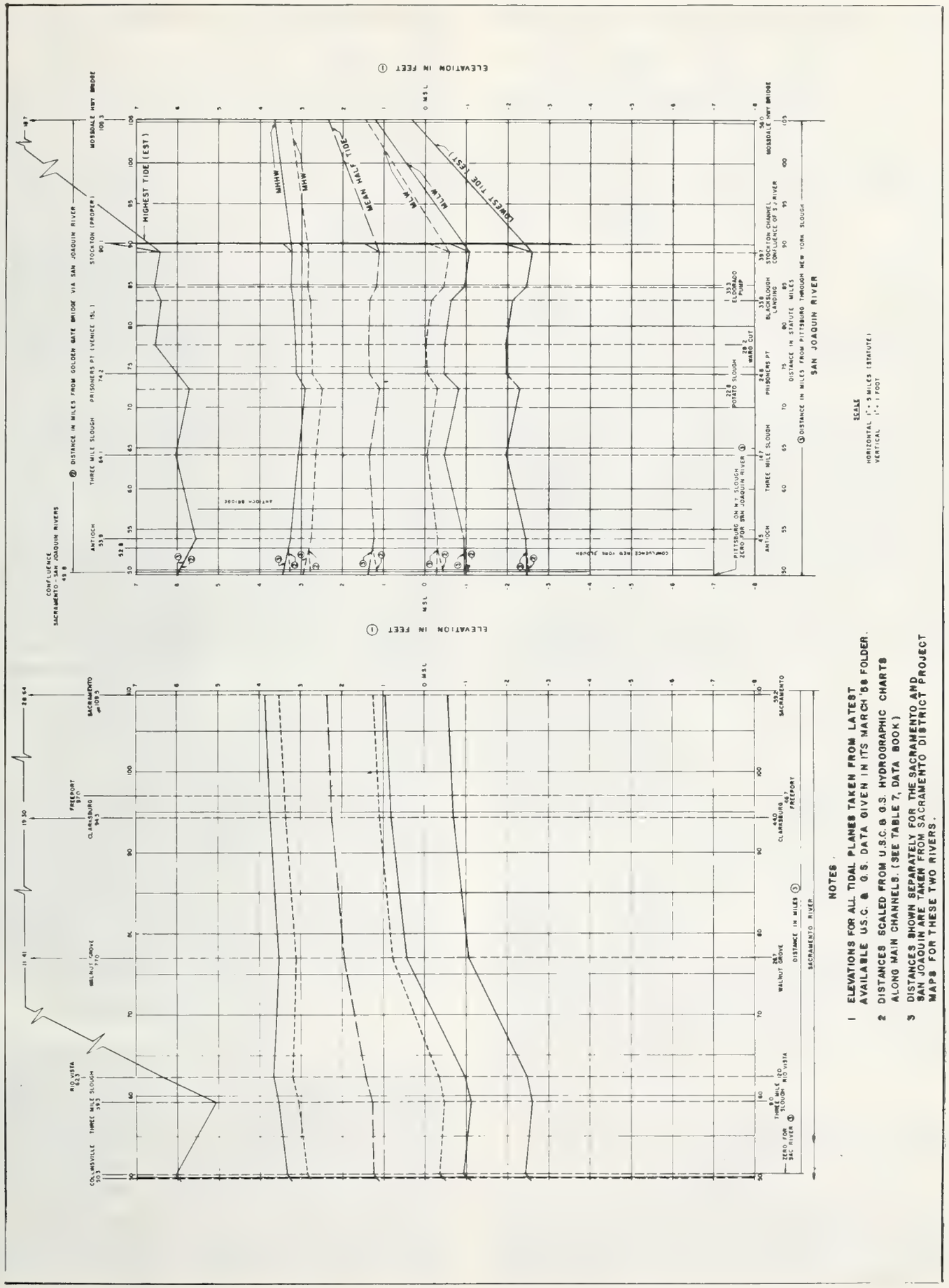

ELEVATION IN FEET ①
MHHW
MHW
MEAN HALF TIDE
MLW
MLLW
LOWEST TIDE (EST.)
HIGHEST TIDE (EST.)
0 MSL
② DISTANCE IN MILES FROM GOLDEN GATE BRIDGE VIA SAN JOAQUIN RIVER
CONFLUENCE SACRAMENTO - SAN JOAQUIN RIVERS
ANTIOCH
THREE MILE SLOUGH
PRISONERS PT (VENICE I.)
STOCKTON (PROPER)
MOSSDALE HWY BRIDGE
PITTSBURG ON M.I. SLOUGH ZERO FOR SAN JOAQUIN RIVER ③
POSEY SLOUGH
WARD CUT
BLACKSLOUGH LANDING
STOCKTON CHANNEL CONFLUENCE OF S. JOAQUIN RIVER
③ DISTANCE IN MILES FROM PITTSBURG THROUGH NEW YORK SLOUGH
SAN JOAQUIN RIVER
SCALE
HORIZONTAL 1" = 5 MILES (STATUTE)
VERTICAL 1" = 1 FOOT
COLLINSVILLE
MID-VISTA
THREE MILE SLOUGH
WALNUT GROVE
CLARKSBURG
FREEPORT
SACRAMENTO
DISTANCE IN MILES ③
SACRAMENTO RIVER
ZERO FOR SAC RIVER ③
NOTES
1 ELEVATIONS FOR ALL TIDAL PLANES TAKEN FROM LATEST AVAILABLE U.S.C. & G.S. DATA GIVEN IN ITS MARCH '58 FOLDER.
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FIGURE 3. — Tidal reference plane charts, Sacramento and San Joaquin Rivers.

TABLE 4.—*Tidal elevations and ranges in San Francisco Bay*

Location	Mean higher high water	Mean lower low water	Mean range	Estimated maximum high water	Estimated minimum low water	Maximum range
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Golden Gate	2.6	−3.1	5.7	5.1	−5.6	10.7
Point San Pablo	2.9	−3.0	5.9	5.6	−5.4	11.0
Selby	3.2	−3.1	6.3	5.9	−5.1	11.0
Benicia	3.3	−2.5	5.8	6.0	−4.5	10.5
Confluence of Sacramento and San Joaquin River	3.3	−1.1	4.4	6.0	−2.5	8.5
Hunters Point	3.3	−3.3	6.6	5.7	−5.6	11.6
San Mateo Bridge	3.5	−3.8	7.3	6.2	−6.3	12.5
Dumbarton Bridge	4.1	−4.3	8.4	6.8	−6.7	13.5
Alviso (South end of Bay)	4.4	−4.6	9.0	6.9	−7.1	14.0

the San Francisco Bay system downstream from Carquinez Strait for the removal of almost 8 million cubic yards of silt from navigation channels. There are in this area a total of 17 authorized projects ranging in depth to 50 feet. The location of these authorized projects is shown on the map in figure 1. The estimated average annual shoaling rates and authorized depths of these projects is shown in table 5.

TABLE 5.—*Authorized navigation projects, project depths, and annual shoaling rates, San Francisco Bay*

Project or facility[1]	Authorized depth	Shoaling rate
	Feet	*Cubic yards*
Mare Island Strait	30 to 26	1,856,000
Pinole Shoal Channel	35	325,000
Oakland Outer Harbor	35	550,000
Oakland Inner Harbor	30 to 25	400,000
Oakland Naval Supply Depot[1]	35	300,000
Oakland Inner Harbor Approach Channel	30	80,000
Alameda Naval Air Station[1]	42	1,000,000
Richmond Outer Harbor	36	400,000
Richmond Inner Harbor	35 to 30	350,000
Islais Creek	35	50,000
San Bruno Shoal	30	275,000
Redwood City Harbor	30	250,000
Main Ship Channel (over the bar)	50	625,000
Point Molate Naval Fuel Depot[1]	45 to 35	95,000
Hunters Point Naval Ship Yard[1]	45 to 30	462,000
Port Chicago Naval Ammunition Depot[1 2]	32 to 15	72,000
Mare Island Naval Ship Yard[1]	30 to 26	752,000
Total		7,842,000

[1] U.S. Naval facility; all others are authorized Corps of Engineers' navigation projects.
[2] Indicated as Concord N.A.D. on figure 1.

The Model

The San Francisco Bay model, which includes most of the area outlined on the map (fig. 1), includes the Bay proper and the lower reaches of tributary streams that are affected by tides to the confluence of the Sacramento-San Joaquin River near Collinsville, and 17 miles of the Pacific Ocean beyond the Golden Gate. The ocean floor is reproduced to the 140-foot depth contour. A considerable area beyond this depth is included to provide space for the primary inflow–outflow headbay. It is one of the largest tidal models in the world. The model is built of reinforced concrete and is of the fixed-bed type. The model floor, except the ocean area, was cast in sections approximately 12 feet square with a screwjack resting on a concrete footing under each corner, to provide means of adjusting for any differential settlement. The linear scale is 1:1,000 horizontally and 1:100 vertically. Salinity in the model is maintained at the same concentration as the prototype. Other scale relationships are shown in table 6.

TABLE 6.—*Scale relationships in model of San Francisco Bay*

Item	Model	Prototype
Velocity	1 ft./sec.	10 ft./sec.
Time	0.6 min.	1 hr.
Do	1 min.	1 hr. 40 min.
Do	14.4 min.	1 solar day (24 hr.)
Do	14.9 min.	1 lunar day (24 hr. 50 min.)
Do	14.9 min.	1 tidal cycle.
Do	7.2 hr.	30 days (1 month)
Discharge	1 c.f.s.	1,000,000 c.f.s.
Do	1 g.p.m.	1,000,000 g.p.m.
Volume	1 cu. ft.	100,000,000 cu. ft.
Do	1 cu. ft.	2,296 acre-feet (approx.)
Area (plan)	1 sq. ft.	1,000,000 sq. ft.
Do	1 sq. ft.	23 acres (approx.)
Do	1 sq. ft.	0.03 sq. mi. (approx.)
Area (cross section)	1 sq. ft.	100,000 sq. ft.

Tides and currents are reproduced by the movement of adjustable valves to vary the inflow (delivered by pumps) and the outflow (by gravity) by which the water surface is raised or lowered at the rate required to simulate tidal action. Inflow of tributary streams is introduced and regulated by weirs and meters by which the fresh water discharge of all significant tributaries can be reproduced to scale.

Shoaling Tests Techniques

Material

Shoaling is simulated in the model by injecting finely ground gilsonite, which is a type of oil shale occurring in nature and having a specific gravity of about 1.04. This material is injected into the model as a slurry containing from 50 percent to as much as 95 percent water. Dyes and other liquid tracers may also be used to an advantage under certain conditions to determine current patterns. Before gilsonite is injected into the model, it must be thoroughly wetted to prevent excess flotation and washed

to prevent excessive discoloration of water in the model.[1]

Model Capabilities and Limitations

It has been found by experience that a hydraulic model, such as that of San Francisco Bay, can be operated so as to reproduce in the model siltation and shoaling patterns similar to those that occur in the prototype. However, it must be recognized that the model cannot reproduce all of the conditions that prevail in nature. The relation between sedimentation and shoaling as it occurs in nature and that which can be reproduced in the model is qualitative only; it being impossible to reproduce and measure in the model to model scale deposition of the same quantity of material as takes place in nature. However, for many conditions the percentage distribution of deposition as it occurs in the prototype can be reproduced in the model, which is entirely satisfactory for most purposes. One reason for the inability to reproduce depositions in the model bearing a direct quantitative relationship to those that occur in nature is the volumetric scale relationship, which is 1:100,000,000. At this scale 1 cc. of material in the model would represent approximately 131 cubic yards in the prototype and if an attempt were made to maintain this relationship the amount of material deposited in navigation channels would be so small as to render reliable measurements almost impossible. Some other limiting factors are:

1. Flocculation, which takes place in nature when silt-laden fresh water comes in contact with salt water, thereby causing precipitation of finely divided suspended solids that are colloidal or semicolloidal in nature, cannot be reproduced in the model because materials used in the model to simulate silt (usually gilsonite) does not flocculate.

2. No practical method has been found to reproduce in the model the action that takes place when material that has been precipitated in shallow areas during calm weather is disturbed and comes back into circulation as a result of wind and wave action.

Type of Tests

Tests conducted in the model fall in two general categories: (1) "base," or verification tests; and (2) "plan," or improvement tests. The object of "base" tests is to reproduce in the model the same distribution of shoals or sedimentation in a specific area of the model that takes place in the prototype. "Plan" tests have as their objective the determination of the effects of improvements on the sedimentation and shoaling pattern. For certain "base" tests, such as dredge spoils disposal areas, prototype data may not be available; therefore, the "base" test consists of observing and measuring the distribution of the material in the model, then modifying certain conditions such as the point at which dredge spoils are dumped; repeating the test, and noting and comparing the effect on shoaling in the model.

[1] A manual on "Operating Procedures for Conducting Shoaling Studies in Model of San Francisco Bay," dated August 1962, prepared by the San Francisco District, Corps of Engineers, provides detail information on the technique of conducting shoaling tests.

Model Tests

Verification

The first step of any shoaling test is to reproduce in the model the same shoaling pattern in the problem area as takes place in nature. (Hydraulic and salinity verifications are assumed to have been accomplished.) This is a trial and error procedure. Perhaps the most critical item is selection of injection points or ranges. The test material (gilsonite) is injected into the model in one or more areas near the problem area at locations that appear to be on the line of travel of sediments deposited in the problem area. Injections are continued over several tidal cycles, then operation of the model continued for several more cycles after injection has been stopped to accomplish complete distribution of the material injected. This operation is usually continuous until there is no appreciable amount of material remaining in suspension. The material is then recovered from each of the separate sections into which the problem area has been previously divided and measured and the percentage of distribution of material so recovered compared with the percentage distribution of shoaling as it occurred in the prototype.

Sometimes very close verification can be obtained. At other times or under other conditions only approximate verification is obtainable. This latter condition occurs more often in a case of deadend channels. Due to the characteristics of gilsonite, it is not transported into these deadend channels to the same extent that silt in the prototype might be so transported. Careful records must be maintained of the total quantities of gilsonite injected at each range and of the location of each injection point, since plan tests require close reproduction of the injection schedule.

Dredge Spoils Disposal Tests

Model tests were conducted to check the merits of present areas used for dumping material dredged from three of the problem areas and of the results to be expected if these areas were relocated closer to areas from which the material was dredged as follows: (1) Oakland Inner and Outer Harbor; (2) Alameda Naval Air Station; (3) Mare Island Strait. Dumping

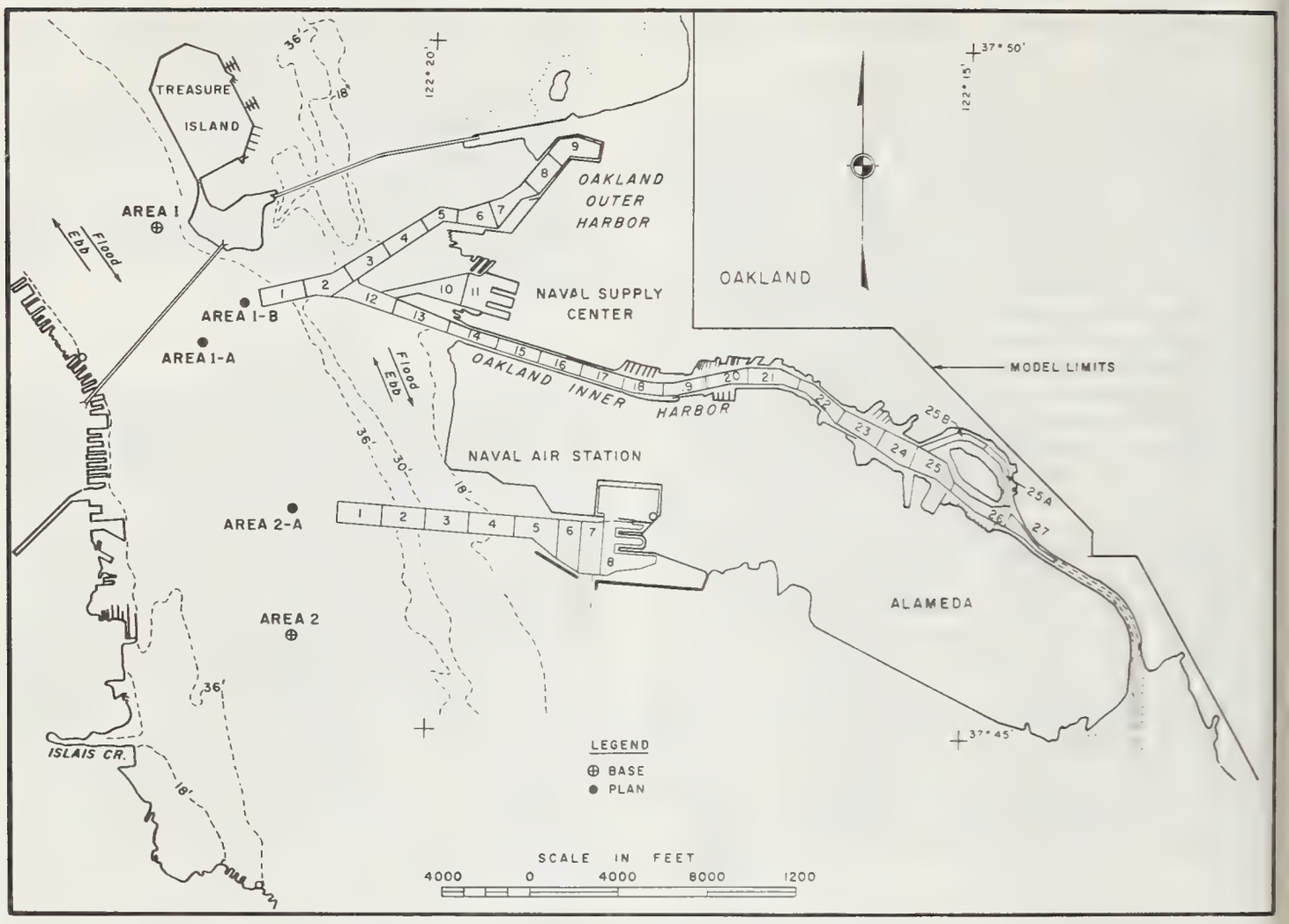

FIGURE 4. — Model of Oakland Harbor and Alameda Naval Air Station.

areas presently used and alternate areas tested in the model for Oakland Inner and Outer Harbor and for the Alameda Naval Air Station are shown in figure 4. Similar information relative to Mare Island Strait is shown in figure 5. "Base" tests consisted of injecting gilsonite at points currently used and observing and measuring its distribution through the Bay system. "Plan" tests consisted of making similar injections at alternate locations; observing and measuring the distribution and comparing the results with the "base" tests to determine the merits of the alternate locations.

Oakland Inner and Outer Harbor

The presently used disposal area of material dredged from this facility is in deep water on the west side of Yerba Buena Island and about 7,000 feet northwesterly from the westerly end of the entrance channel to these facilities, designated as area 1 on the map (fig. 4). Dredge spoils are dumped at this place at any time without regard to the direction of flow of the tide. For the "base" tests a continuous injection amounting to a total of approximately 150,000 cc. of gilsonite was made at this point. In order to determine the feasibility of moving the dredge spoil disposal area nearer to the project and thereby reducing travel time and distance for the dredges, two alternate locations designated as areas 1A and 1B on the map (fig. 4), and located nearer the project area, were tested by injecting similar amounts of gilsonite in these locations. The results are shown in table 7.

Very little difference occurred in the distribution of dredge spoils dumped at the three areas considered (table 7). Tests indicate that, on an average, less than 4 percent of material dumped at any of these locations is redeposited directly in navigation channels in the Bay system, and approximately 30 percent of the dredge spoils is carried directly to sea through the Golden Gate. It also appears that approximately 65 percent

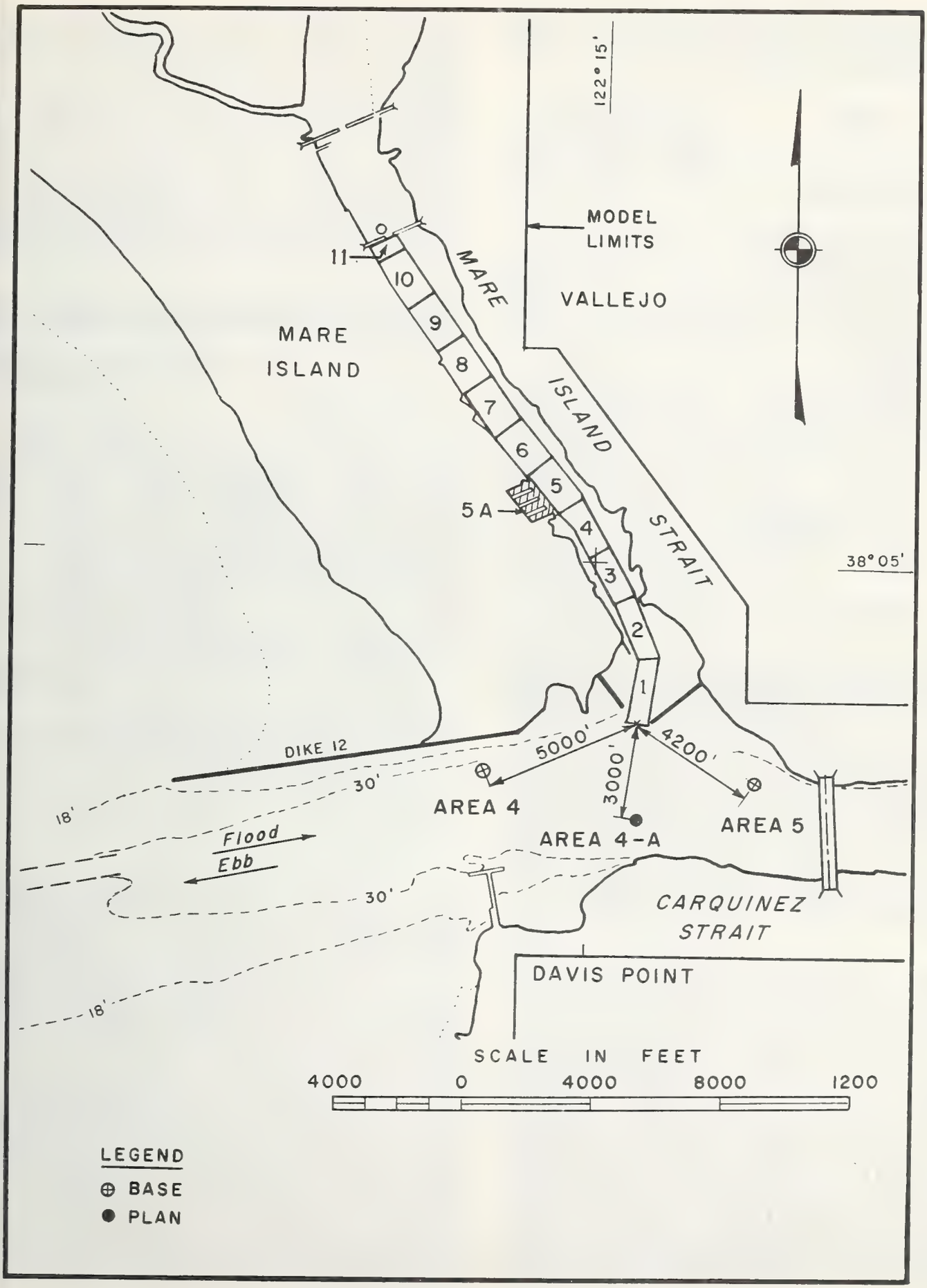

FIGURE 5. — Model of Mare Island Strait.

TABLE 7.—*Comparison of "base" and "plan" tests of dredge spoil disposal areas for the Oakland-Alameda group projects*

Projects	Gilsonite deposits of total quantity (cc.) injected		
	Area 1 (base)	Area 1A (plan)	Area 1B (plan)
	Pct.	*Pct.*	*Pct.*
Richmond Harbor Channels	0.73	0.65	0.60
Oakland-Alameda Group projects	.84	.37	1.02
Hunters Point and Islais Creek	.08	.11	.18
San Bruno Shoal, South Bay	.35	.30	.44
San Francisco Pier Slips	.67	.70	.76
Pinole Shoal, San Pablo Bay	.....	.10	.12
Richardson Bay Channel	.12	.16	.16
Main Ship Channel over Bar	.86	.60	.60
(1) Subtotal, navigation facilities	3.65	3.99	
(2) Richardson Bay, outside of Channel (measured)	.45	.64	.62
San Francisco Bar, south	7.78	7.89	5.84
San Francisco Bar, north	3.18	2.45	2.53
(3) Subtotal outside of Golden Gate minus bar channel	34.72	23.18	29.36
(4) Subtotal interior Bay, outside of navigation facilities (estimated) Minus (2) Richardson Bay)	61.18	73.19	66.14
Totals (1), (2), (3), and (4)	100.00	100.00	100.00

of the dredge spoils is redeposited in parts of the Bay outside of navigation projects. It is probable that a considerable part of this material may be resuspended by wind and wave action. An indeterminate amount of the resuspended material would be deposited ultimately in navigation channels. A considerable part would also be carried to sea through the Golden Gate. Tentative conclusions derived from these tests are that alternate or "plan" areas can be used for disposal of dredge spoils without detrimental effect, and a worthwhile savings thus accomplished through reduction of haul distance.

Alameda Naval Air Station

Test procedure for this facility were similar to those for the Oakland Inner and Outer Harbor. In figure 3, the presently authorized spoils disposal area designated as area 2 on the map is located some 7,000 feet southerly from the westerly end of the approach channel and the alternate or "plan" area designated as area 2A is located on the prolongation of the centerline of the approach channel. The use of this latter area would result in a decrease of approximately 2 miles in the haul distance from the project to the spoils disposal area. The results of the tests of these two areas are shown in table 8.

There is very little difference in the dispersion pattern of material dumped in either of the two areas (table 8). In fact, the tests indicate that the plan area (area 2A) has a slight advantage. The distribution pattern is very similar to that for the Oakland Inner and Outer Harbor. About 4 percent of the material deposited in navigation channels and 25 percent to 30 percent is carried out to sea through the Golden Gate. Tentative conclusions are that the plan area (area 2A) can be adopted without detrimental results, and an appreciable savings in dredging costs accomplished through reduction of hauling distance.

TABLE 8.—*Comparison of "base" and "plan" tests for dredge spoil disposal areas for Alameda Naval Air Station*

Location	Gilsonite deposits	
	Area 2 (base)	Area 2A (plan)
	Pct.	*Pct.*
Richmond Harbor channels	0.45	0.64
Oakland-Alameda group projects	.26	.40
Hunters Point and Islais Creek	.15	.15
San Bruno Shoal, South Bay	1.63	1.02
San Francisco Pier slips	1.09	.88
Pinole Shoal, San Pablo Bay	.12	.22
Richardson Bay channel	.09	.11
Main Ship channel over bar	.23	.13
(1) Subtotal in navigation facilities	4.02	3.55
(2) Richardson Bay, outside of channel (measured)	0.37	0.45
San Francisco Bar, south	5.10	3.63
San Francisco Bar, north	1.72	1.65
(3) Subtotal outside of Golden Gate, minus Bar channel	31.09	21.78
(4) Subtotal interior, outside of navigation facilities (estimated) minus (2) Richardson Bay	64.52	74.22
Totals (1), (2), (3), and (4)	100.00	100.00

Mare Island Strait

Areas currently used for disposal of dredge spoil and the area tested in the model for this facility are shown in figure 5. At this facility dredge spoils are dumped during and at ebbtide at area 4, located downstream from the mouth of the Strait; and on floodtide at area 5, located upstream from the mouth of the Straits. Two separate "base" tests were conducted at this location, one during which injections were made only on the ebb at area 4, and one during which injections were made only on the floodtide at area 5. In the "base" tests approximately 50,000 cc. of gilsonite were released in each area. For the plan test at area 4A, injections were made continuously through both the ebb- and floodtide and approximately 100,000 cc. of gilsonite, or twice the quantity used for each of the individual plan tests, was injected. The distribution pattern of each of the "base" tests and of the

"plan" test and a comparison of results is given in tables 9, 10, 11, and 12.

TABLE 9.—*Spoil disposal area 4 (base), deposition within and without navigation channels and slips*

Project	Gilsonite deposits	
	Cubic centimeters	Percent of total cc. injected
Pinole Shoal, San Pablo Bay	5,755	11.5
Mare Island Strait	2,410	4.8
Suisun Bay Channel	[1]50	.1
Total	8,215	16.4
Outside of Golden Gate, no evidence of deposit	.....	.....
Central Bay, very little evidence of deposit	.....	.....
Interior Bay, outside of projects	41,785	83.6
Totals	50,000	100.0
Measurements made of gilsonite, deposits in limited areas for comparison with results of subsequent tests, as follows:		
(1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour	980	2.0
(2) ½-square-mile area in Carquinez Strait from Benicia shore to 18-ft. depth contour	500	1.0

[1] Estimated.

TABLE 10.—*Spoil disposal area 5 (base), deposition within and without navigation channels and slips*

Project	Gilsonite deposits	
	Cubic centimeters	Percent of total cc. injected
Pinole Shoal, San Pablo Bay	1,140	2.5
Mare Island Strait	2,870	6.2
Suisun Bay Channel	230	.5
Total	4,240	9.2
Outside of Golden Gate, no evidence of deposit	.....	.....
Central Bay, very little evidence of deposit	.....	.....
Interior Bay, outside of projects	41,760	90.8
Totals	46,000	100.0
Measurements made of gilsonite deposits inlimited areas for comparison with results of other gase and plan tests, as follows:		
(1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour	5,200	11.3
(2) ½-square-mile area in Carquinez Strait from Benicia shore to 18-ft. depth contour	2,415	5.3

TABLE 11.—*Spoil disposal area 4A (plan), deposition within and without navigation channels and slips*

Project	Gilsonite deposits	
	Cubic centimeters	Percent of total cc. injected
Pinole Shoal, San Pablo Bay	5,075	5.4
Mare Island Strait	4,655	4.9
Suisun Bay Channel	[1]300	.3
Total	10,030	10.6
Outside of Golden Gate, no evidence of deposit	.....	.....
Central Bay, very little evidence of deposit	.....	.....
Interior Bay, outside of projects	84,720	89.4
Totals	94,750	100.0
Davis Point (Union Oil Co. pier):		
Area 1 (530,000 sq. ft.)	110	.1
Area 2 (230,000 sq. ft.)	230	.2
Measurements made of gilsonite deposits in limited areas for comparison with results of other base and plan tests, as follows:		
(1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour	7,415	7.8
(2) ½-square-mile area in Carquinez Strait, from Benicia shore to 18-ft. depth contour	3,880	4.1

[1] Estimated.

TABLE 12.—*Comparison of test results for base and plan tests of dredge spoil disposal areas for the Mare Island Strait project*

Project	Distribution			
	Area 4 base	Area 5 base	Area 4+5 base	Area 4A plan
	Percent	*Percent*	*Percent*	*Percent*
Mare Island Strait	4.8	6.2	5.5	4.9
Pinole Shoal	11.5	2.5	7.2	5.4
Suisun Bay Channel[1]	.1	.5	.3	.3
Totals	16.4	9.2	13.0	10.6
Outside of Golden Gate	0	0	0	0
Central Bay	0	0	0	0
Interior Bay, outside of projects	83.6	90.8	87.0	89.4
Totals	100.0	100.0	100.0	100.0
Northwest shore of Suisun Bay (limited area of 2 sq. mi.)	2.0	11.3	6.4	7.8
Benicia Shore (limited area of ½ sq. mi.)	1.0	5.3	3.0	4.1
Davis Point (Union Oil Co. pier):				
Area 1 (530,000 sq. ft.)	.....	.....	.....	.1
Area 2 (230,000 sq. ft.)	.....	.....	.....	.2

[1] Estimated.

From the foregoing tabulations practically none of the material injected in the disposal areas of Mare Island Strait appears to be car-

ried to sea and a considerable part of it actually appears to be transported up Carquinez Strait and into Suisun Bay. The test of the "plan" (area 4A) indicates slightly less redeposition in navigation channels than do either of the areas currently used. Since the annual shoaling rate and dredging rate in Mare Island Strait, including the Naval Ship Yard, is approximately 2,600,000 cubic yards, this indicates that use of the "plan" area might result in a decrease of about 16,000 cubic yards a year in the shoaling rate in Mare Island Strait and a slight reduction in Pinole Shoal.

Tentative conclusions are that the "plan" area (area 4A) can be used without detrimental results and that its use might result in some decrease in the shoaling rate in Mare Island Strait. Furthermore, a slight reduction in hauling distance could also be accomplished.

Model Tests of Shoaling in Navigation Projects

Model tests were conducted to ascertain the effect of various improvements consisting chiefly of dikes and training walls on shoaling in navigation projects. The results of some of the tests of plans for the Alameda Naval Air Station, where the average shoaling rate is about 1 million cubic yards a year, are considered typical of results that might be anticipated from similar plans elsewhere. The results of tests of 11 plans are shown in table 13. Plans and channel sections are shown on figure 6.

TABLE 13.—*Shoaling test results after various improvements, Alameda Naval Air Station*

Channel section No.	Shoaling distribution after improvements	Change in shoaling for plan—										
		1	2	3	4	5	6	7	8	8a	9	10
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	0.6	+33	+33	−33	0	+33	+33	0	0	+33	−33	−33
2	0.9	0	+80	+60	+80	+80	+680	+80	+100	+60	+80	+80
3	0.7	0	+25	+150	+75	+125	+3,375	+75	+25	+50	+50	−20
4	10.1	+38	+113	−24	+55	+18	+85	+40	+18	−11	+16	+25
5	30.9	+6	−19	−36	−14	−31	−69	−20	−76	−50	−32	−44
6	17.6	−10	−41	−44	−31	−23	−65	−9	−76	−54	0	−20
7	16.4	−30	−52	−53	−39	−26	−67	+21	−83	−64	−15	−10
8	22.8	−55	−69	−66	−58	−47	−79	+7	−93	−76	−23	−33
Total	100	−11	−25	−44	−23	−25	−21	+2	−69	−53	−15	−23

Model tests indicate that there is a direct relation between the volume of water (the tidal prism) flowing into or through a tidal basin or across an excavated navigation channel and the amount of silt deposited in such a facility. In the facility under consideration under present conditions, a considerable volume of water flows through the basin on both the flood- and ebb-tides. On the floodtide, water enters the basin from the west and a part of it passes out through the 750-foot gap in the breakwater on the south side of the docking area. On the ebbtide, the reverse condition prevails. Closing the gap, as indicated for plan 1, which reduced the tidal prism to the volume within the docking area represented by that area and the tidal range, produced a net reduction of 11 percent in siltation throughout the project, with the greatest reduction taking place in the part protected by the breakwater (sections 6, 7, and 8) but with increases in some sections of the approach channel (sections 1, 4, and 5). During the test a large eddy formed over the navigation channel seaward of the area protected by the breakwater, and a considerable volume of water that would normally flow into the sheltered area passed over sections 4 and 5, which accounts for the increase in shoaling in these sections.

Plan 2, which consisted of a closure of the 750-foot gap in the breakwater and a 2,000-foot extension of the breakwater, produced a still further decrease in net total shoaling with a large decrease in the area protected by the breakwater but with an increase in shoaling in the unprotected section of the channel (sections 1 through 4). In this test an eddy extending across the approach channel and to the shore immediately north of section 4, developed during the test. Plan 3, which includes all features of plan 2 plus a dike extending 1,300 feet southerly from the shore north of section 4, affected the pattern of this eddy and accomplished a further reduction in total shoaling but resulted in a further increase in depositions in the approach channel to the west, sections 1, 2, and 3. Plans 4, 5, and 6 are based on the same concept as plan No. 3; that is, deflection of the eddy which appeared to be responsible for depositions in that section of the channel not protected by the breakwater.

Plans 7 and 9 were tested to determine the possibility of accomplishing a worthwhile reduction without closing the 750-foot gap in the breakwater southerly from the docking area, since this opening is desired in order to provide a passage for seaplanes. The results of these plans, one of which resulted in a net increase of 2 percent in total depositions, proved conclusively that closure of this gap is a key factor.

Plan 8, which reduces the access area to the

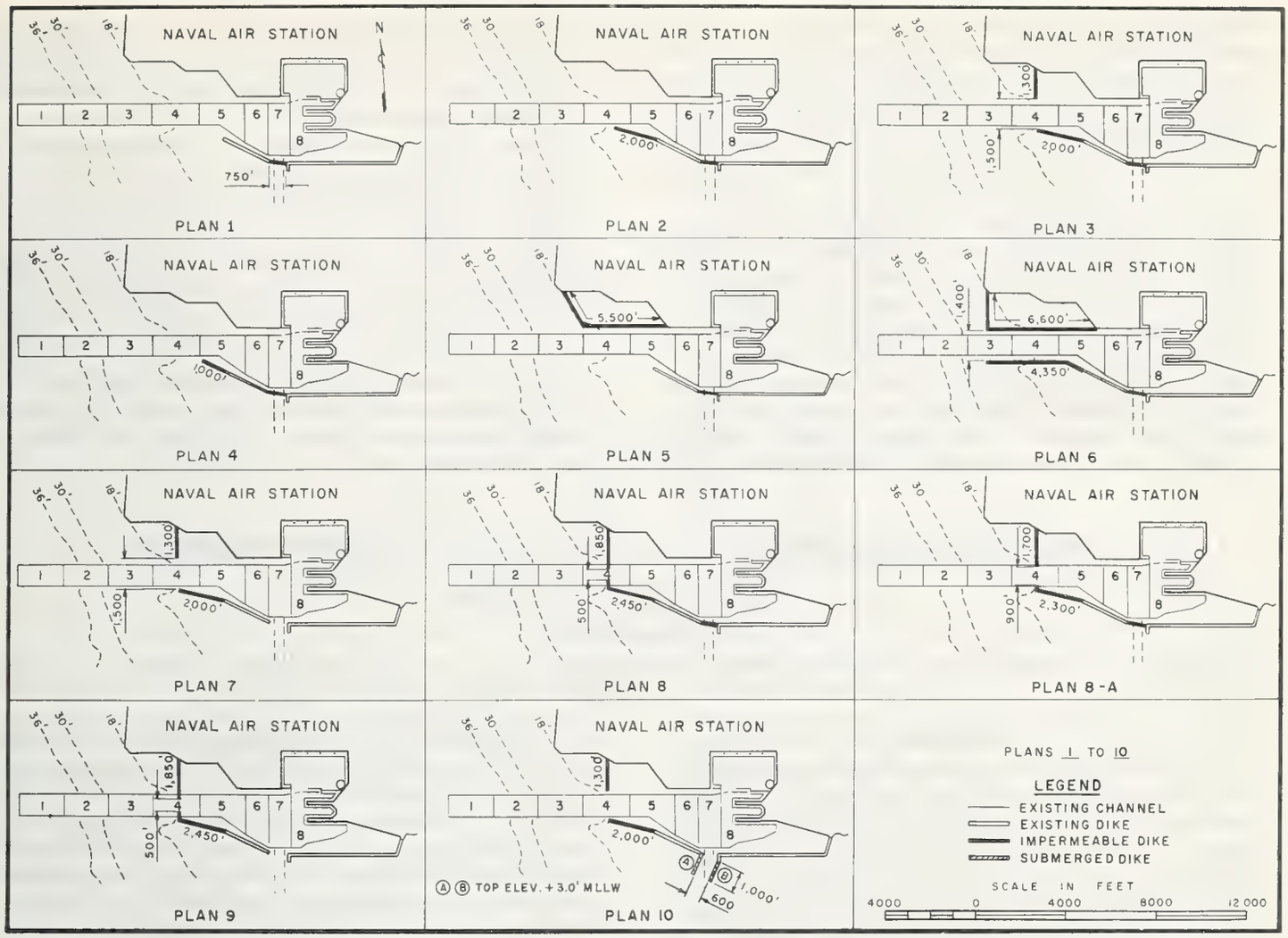

FIGURE 6. — Models of plans 1 to 10, to study shoaling in navigation projects.

eastern section of the project where about 90 percent of the shoaling takes place to a cross section 500 feet wide, proved to be the most effective in that the net reduction in shoaling was 69 percent. This plan demonstrates conclusively the theory that there is a direct relation between the amount of water flowing into or across an improved channel or docking area and siltation in the facility. This plan eliminated substantially all of the eddy currents that had resulted in great increases in deposition in sections 1 through 4, although it did show some increase in sections 2, 3, and 4. This increase is accounted for by the fact that the restriction in the approach to the sheltered area probably resulted in a slight buildup westward of the opening and also probably caused a slightly greater amount of water to pass across sections 2 and 3 and that part of section 4 westward of the entrance than would take place under existing conditions.

Plan 10 was tested to determine the effect of deflecting dikes that would divert water flowing directly through the 750-foot opening in the breakwater on an ebbtide. It was considered probable that considerable part of the silt carried into the docking area through this opening on an ebbtide traveled as a bedload movement or at least for the lower part of the stream, and that the dikes might serve to deflect the currents to the west. The results indicate some improvement; however, a comparison with plan 9 cast doubt on the effectiveness of this plan.

Discussion

The results of the program of model tests of hopper dredge spoil disposal reported herein are adequate to evaluate the current practice of disposal of spoil from the standpoint of effects on main navigation channels, side channels, and harbors and on slips and small boat basins while dredging operations are in progress. The present practice of "free-dumping" of dredge spoil beyond the limits of the channel and the disposal areas themselves are satisfactory. However, since the amount of material that deposits

in navigation facilities is insignificant, and with very little differences resulting from the disposal areas currently being used for the newly located areas, it is concluded that "plan" areas located closer to the dredging projects should be utilized in future dredging operations. Thus, by reducing the haul distance, considerable savings can be effected in the dredging operations. Studies have not progressed far enough to determine whether a change in present practice to a more expensive practice of controlling spoil, such as the sump rehandler dredging technique (to insure that none of the dredge spoils returns to the waterway) is warranted. When consideration is given to the large amount of spoil that deposits in shallow areas of the Bay, from whence large amounts are resuspended by wave action and carried by tidal currents into the various navigation facilities, rehandling and disposal in diked areas may be warranted for some projects.

With regard to plans of improvement to reduce shoaling, additional tests and studies are required for further substantiation of results indicated and for the development of details of plans that might be recommended. Also several other problem areas appear to be susceptible of analysis and possible solution by model tests. Results to date indicate that the model tests will result in material savings in maintenance dredging costs, and in a much better understanding of shoaling problems in the San Francisco Bay system.

CHANNEL DEPTH AS A FACTOR IN ESTUARINE SEDIMENTATION

[Paper No. 72]

By HENRY B. SIMMONS, *chief, Estuaries Section, Rivers and Harbors Branch, Hydraulics Division, U.S. Army Engineer Waterways Experiment Station, Vicksburg*

Abstract

In most estuaries in the United States that have been improved for navigation by large oceangoing vessels, drastic changes have taken place in both annual shoaling rates and in the locations of major shoal areas as the channels were progressively deepened. Numerous problems that affect channel maintenance have arisen because of these changes, and some of these problems have now reached a critical state.

This paper presents a case history of Savannah Harbor to illustrate how shoaling rates and patterns have been changed by progressive changes in channel depth. The paper also discusses the more important reasons for such changes and suggests remedial measures that appear to offer both technical and economic solutions to certain of the problems thus created.

When the early settlers of this country reached the Atlantic and Gulf coasts in their sailing ships, they discovered that the natural estuarine channels afforded ready access to the interior for sufficient distances to locate suitable sites for towns and farms and to provide sheltered anchorages for their ships. Cities such as Charleston, Savannah, Mobile, and many others were founded on the first terrain encountered which was sufficiently high above high tide for development. Although the natural estuarine channels were usually characterized by entrance bars and interior crossings, some having controlling depths of perhaps 6 to 12 ft. at low tide, such depths were usually adequate for safe navigation by the oceangoing ships of that era. Furthermore, after having spent weeks or even months at sea since leaving the last port of call, the ship captains were not disturbed by a delay of several hours waiting for high tide to navigate an especially troublesome bar or crossing.

Most of the natural estuaries discovered by these early settlers were apparently in a state of essential equilibrium between the rate of supply of sediments from both fluvial and oceanic sources and the ability of the currents generated by tides and freshets to remove such sediments and carry them to sea. The environment in which these natural estuaries existed was a comparatively simple one, since there were essentially no agricultural developments to accelerate erosion and no attempts had been made to alter the physical characters of the channels or the hydraulic characters of flows therein. Because of the usual semicircular entrance bars, which had crest elevations only a few feet below low tide, it is probable that density currents played only a very minor role in the sedimentation processes of these natural estuaries. Although the locations and crest elevations of bars and crossings were probably changed drastically from time to time by severe storms and major freshets, it appears likely that the controlling long-term conditions were restored fairly quickly after such events.

With the advent of steam power for ocean-going ships, the ship designers quickly discovered that increased vessel size and draft paid handsome dividends in terms of lower construction and operating costs per unit of cargo capacity; that these factors are still important today is most dramatically evidenced by the modern supertanker. As the sizes and drafts of ocean-going ships increased, the natural depths of estuaries used for navigation were no longer adequate and a chain of actions and reactions was begun that is still in progress at the present time.

The first attempts to improve navigating conditions in tidal waterways consisted largely of the removal of snags and other obstructions, followed by the closure of secondary channels to confine flows to major channels, and then by the construction of dikes and jetties to increase depths over interior crossings and entrance bars. Maximum depths sought through such improvement measures were usually 15 to 20 ft. at low tide. With a minor amount of initial and maintenance dredging, the engineers concerned were largely successful in obtaining the rather moderate depths desired. Repetitive maintenance dredging was generally not a major problem, since the environments of the waterways involved had still not been altered to a significant degree. However, as increasing vessel drafts required greater and greater depths, which were obtained and maintained largely by dredging — supplemented in some cases by channel contraction works and more extensive entrance jetty systems — it became increasingly obvious to the engineers concerned that important environmental factors had been altered to such extent that many of the improved estuaries bore little resemblance either physically or hydraulically, to those that existed in the natural state. It is therefore not surprising that principles of improvement that at first yielded highly satisfactory results became less and less effective as channel depths were progressively increased.

The various types of estuaries encountered, the hydraulic and salinity conditions characteristic of each type, and the effects of upland discharge in controlling estuary type were discussed by the author in 1955.[1] Some of the shoaling characteristics of the various types of estuaries and principles of improvement plans designed to reduce shoaling were discussed by the author and E. A. Schultz in 1957.[2] This present paper is concerned primarily with the effects of channel depth as a varying environmental factor on the amount of repetitive shoaling in estuaries, and more particularly on the locations of major shoal areas within estuaries.

At first thought it might appear that the location of the major shoal area within an estuary would not be of major importance from an economic viewpoint, provided the dredging frequency and the quantity of dredging per unit time were the same. However, because suitable areas for disposal of dredged soil are rapidly being filled, which necessitates longer discharge lines to reach remote areas and in some cases booster pumps to maintain efficiency, the unit cost of dredging may vary over a wide range, depending on the location of the shoal area being dredged. As an example, the unit cost of dredging various shoal areas within one navigation project familiar to the author ranges from as little as $0.15 to as much as $1.00 per cu. yd. This wide range depends almost entirely on whether there are spoil disposal areas immediately adjacent to the dredging site that require little or no retention dike construction or whether the spoil must be placed in remote disposal areas that require the construction of elaborate retention dikes to prevent the spoil from running back into the channels being maintained or from flowing onto and damaging adjacent properties.

The effects of channel depth on the shoaling regimen of an estuary can probably be best illustrated by an actual case history; that is, by taking a given estuary and developing a record of the rates and patterns of shoaling at various stages of its development. The author has selected Savannah Harbor for this purpose for three principal reasons. First, Savannah Harbor is characteristic of the partly mixed estuary, which is the most common type in the United States that has been improved for deep-draft navigation; second, Savannah Harbor is a relatively simple type of estuary from the standpoint of channel complexity; and third, the engineers who have been responsible for the development and maintenance of Savannah Harbor have been more painstaking than most in keeping good records, so that the determination of reasonably accurate shoaling rates and patterns for previous channel depths has been possible. Also, two rather comprehensive hydraulic model investigations of Savannah Harbor have been carried out by the Waterways Experiment Station during the past 20 years, and information gained during these investigations has contributed much to an understanding of the present and former hydraulic and shoaling regimens of the harbor.

Savannah Harbor (fig. 1) comprises the

[1] SIMMONS, H. B. SOME EFFECTS OF UPLAND DISCHARGE ON ESTUARINE HYDRAULICS. Amer. Soc. Civil Engin. Proc. Paper 792. 1955.

[2] SIMMONS, H. B., and SCHULTZ, E. A. FRESH WATER-SALT WATER DENSITY CURRENTS, A MAJOR CAUSE OF SILTATION IN ESTUARIES. U.S. Army Corps of Engineers Committee on Tidal Hydraul. Tech. Bul. 2. 1957.

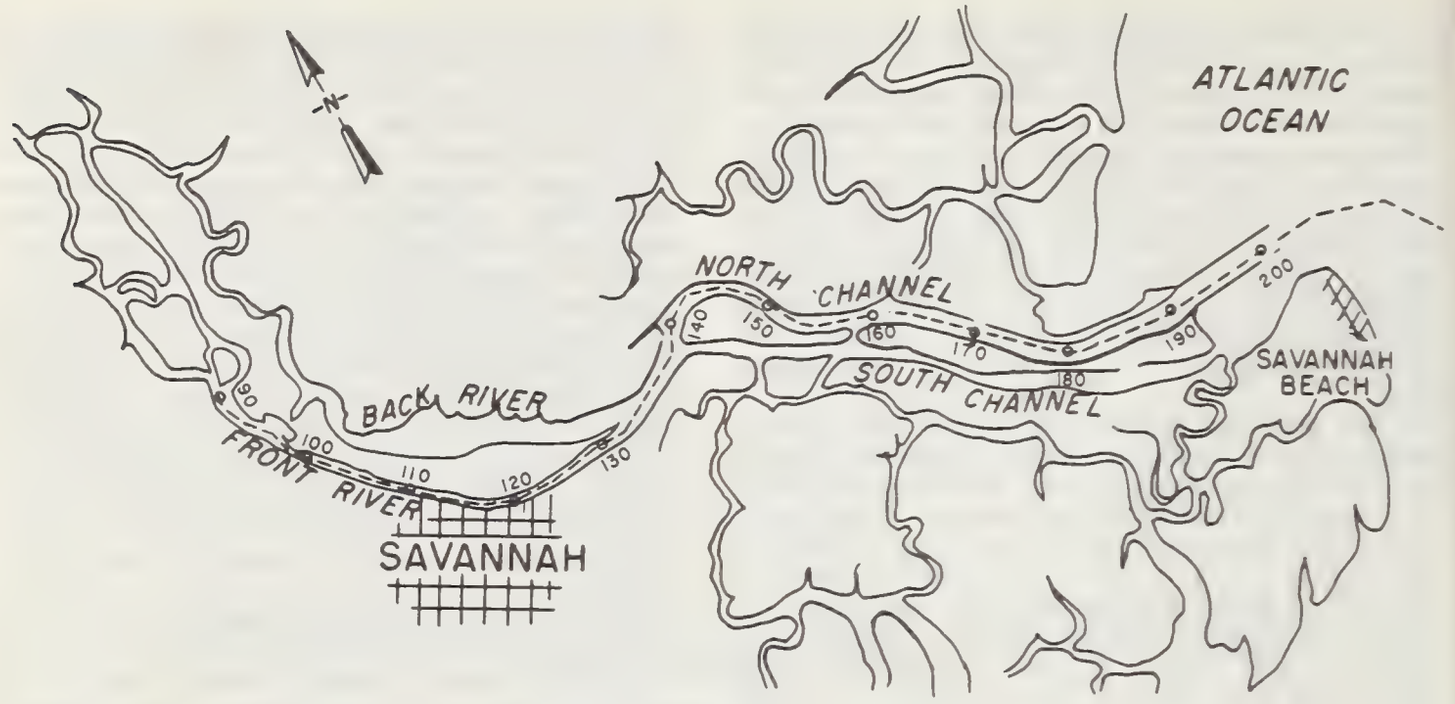

FIGURE 1. — Location map, Savannah Harbor.

lower 23.3 miles of the Savannah River, which is located on the South Atlantic coast of the United States and forms the boundary between Georgia and South Carolina, plus 7.5 miles of bar channel between deep water in the Atlantic Ocean and the entrance jetties. The city of Savannah is located 16.1 miles upstream from the entrance to the harbor, and the improved channel extends another 7.2 miles upstream from the city. The navigation channel follows North Channel from the ocean to the head of South Channel and then follows Front River to the head of navigation at channel station 82. The present navigation project for the harbor calls for a channel 500 ft. wide and 36 ft. deep at mean low water (m.l.w.) from deep water in the Atlantic Ocean to channel station 190; thence 400 ft. wide and 34 ft. deep at mean low water to channel station 94; thence 200 ft. wide and 30 ft. deep at mean low water to the head of navigation at channel station 82. The two jetties at the entrance to North Channel, which are about 2,500 ft. apart, were constructed between 1890 and 1896.

The mean range of tide at the entrance to Savannah Harbor is about 6.8 ft., that at Bull Street Gage (station 120) is about 7.4 ft., and that at the head of the navigation project is about 8.0 ft. Maximum tidal current velocities in the navigation channels are 5 ft. per sec. during spring tides. Prior to construction of Clark Hill Dam upstream from Augusta, Ga., the fresh-water discharge of the Savannah River ranged from a normal low of about 4,000 c.f.s. to a maximum of about 350,000 c.f.s. Since construction of the dam, the fresh-water discharge has been effectively regulated at about 7,000 c.f.s.; however, local heavy rains in the watershed downstream from the dam have increased the discharge at Clyo, Ga. (the head of tidal influence), to about 25,000 c.f.s. for periods of several days. For normal conditions of tide and fresh-water discharge, salt water having a salinity of about 5 parts per thousand (p.p.t.) penetrates upstream to about channel station 109 at high-water slack of the tidal current and retreats to about channel station 129 at low-water slack.

As stated previously, Savannah Harbor is characteristic of the partly mixed estuary, in that the vertical mixing of salt and fresh water is not complete over the length of salt-water intrusion. In this type of estuary, surface salinities are appreciably less than bottom salinities and a significant part of the vertical salinity difference occurs within a few feet of the depth. This transition zone may be considered to be an "interface," even though it is not well defined, because the net flow over a complete tidal cycle is downstream above the transition zone and upstream below the transition zone. The vertical distribution of flow in Savannah Harbor for conditions of mean tide and a normal fresh-water inflow of about 7,000 c.f.s. is shown in figure 2. The numerical values on this figure represent the percentage of total flow over a complete tidal cycle that was downstream or ebb at each point of measurement. A value of 50 indicates that upstream and down-

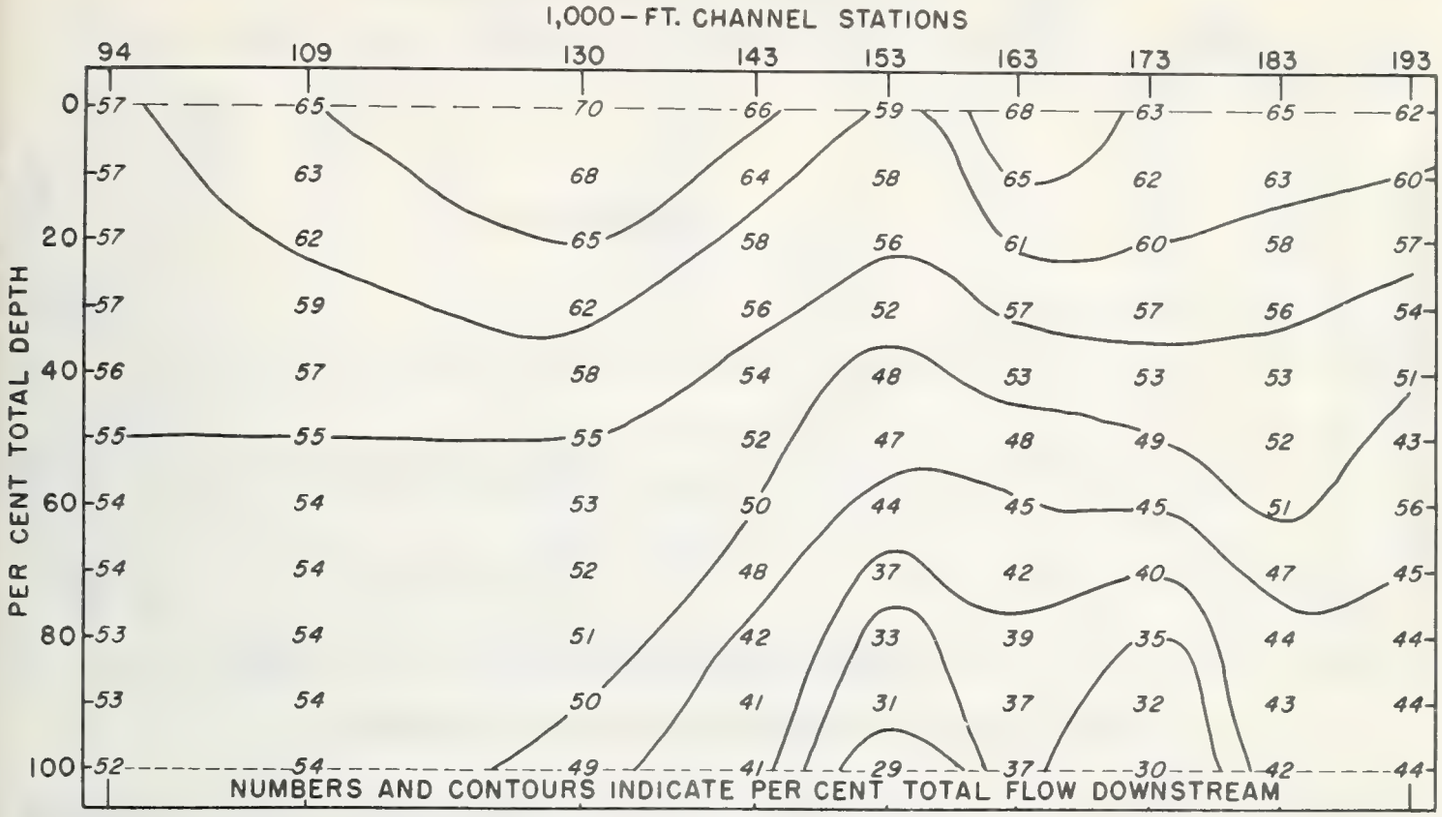

FIGURE 2. — Distribution of flow in Savannah Harbor.

stream flows were exactly in balance; values greater than 50 indicate that downstream flows predominated over upstream flows; values less than 50 indicate that upstream flows predominated over downstream flows.

Examination of figure 2 shows that the direction of net bottom flow in Savannah Harbor is upstream from the harbor entrance to about channel station 125 and is downstream from this location to the upper end of the navigation project. Bottom upstream predominance reaches a maximum in the vicinity of station 153, where the floodflow is about 71 percent of the total flow over a tidal cycle, and the ebb is only about 29 percent. The elevation of the transition zone or interface between net upstream and net downstream flow is located at about 70 percent of the channel depth at station 140, about 35 percent of the depth at station 155, and about 50 percent of the depth at station 190. The direction of net flow at the surface and at all other elevations above the interface is downstream. Upstream from the limit of salt-water intrusion, the direction of net flow is downstream at all depths, and the degree of downstream predominance at any location in this reach is an indication of the relation between the tidal discharge and the fresh-water discharge.

The sediments that deposit in Savannah Harbor to form shoals consist almost entirely of silts and flocculated clays, and it is obvious that the transportation and deposition of these lightweight sediments are greatly influenced, if not controlled completely, by the flow distribution just described. If a given sediment particle entering the upstream end of the harbor is of such nature as to deposit first in the fresh-water region, it would likely be resuspended during each strength of velocity and thus be moved progressively downstream to the vicinity of station 125, beyond which point the direction of the net flow is upstream. On the other hand, should the nature of the particle be such that it would remain in suspension until deposited through flocculation or related processes, it would likely first deposit at some location well downstream from station 125 where salinities are relatively high. Again the particle would likely be resuspended during each strength of current, but in this case it would be moved progressively upstream to the vicinity of station 125 by the net bottom upstream flow. Therefore, in both of these assumed cases, there is a net force tending to concentrate sediment in the vicinity of station 125, since the direction of net bottom flow is toward this general location from both directions.

Figure 3 shows the relation between bottom flow predominance in Savannah Harbor and the

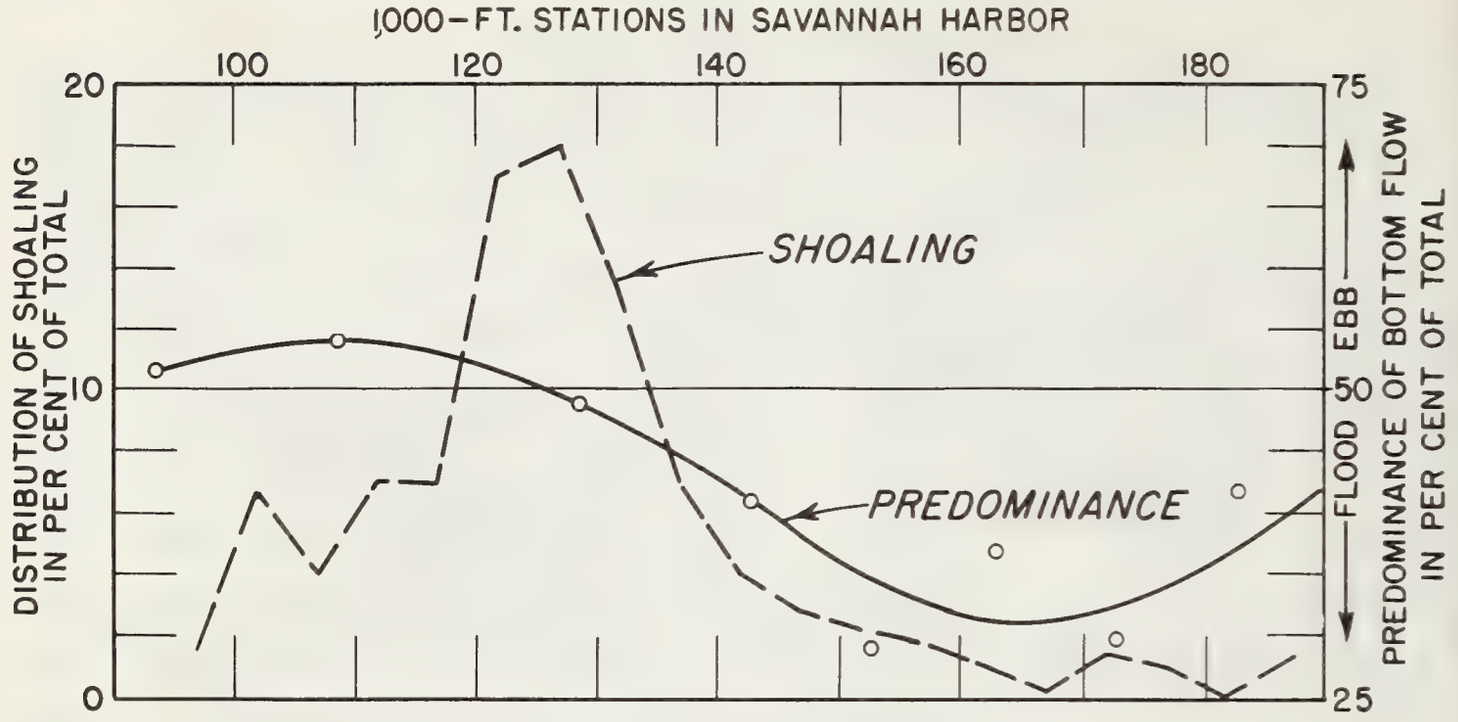

FIGURE 3. — Relation of shoaling and predominance of bottom flow.

average annual distribution of shoaling in the harbor. The solid line in figure 3 is a replot of the bottom flow predominance values from figure 2, again plotted against 1,000-ft. channel stations. A value of 50 percent again indicates that upstream and downstream flow is in balance, while values in excess of 50 percent indicate downstream predominance and values less than 50 percent indicate upstream predominance. The nodal point, or the location where upstream and downstream flow at the bottom is in balance, is at about station 125 as previously described in the preceding paragraph. The dashed line in figure 3 shows the average annual distribution of shoaling in Savannah Harbor for the years 1953 and 1954, during which period the channel depths and widths throughout the harbor were as just described.

In this plot, the harbor has been divided into 5,000 ft.-long sections, and the value plotted in the center of each section represents the percentage of total shoaling of the harbor that occurred in that section. Of the total shoaling 35 percent occurred between stations 120 and 130 or within the 2-mile-long channel reach encompassing the nodal point for bottom flow predominance. In addition, 20 percent of the total occurred in the adjacent 2-mile-long downstream reach, and 14 percent of the total occurred in adjacent 2-mile-long upstream reach. Therefore, from a total improved channel length of about 31 miles, about 69 percent of the total shoaling occurred in a 6-mile-long reach that brackets the nodal point for bottom flow predominance. Thus, more than two-thirds of the total annual shoaling occurs in less than one-fifth of the total length of the dredged channel.

Both the annual shoaling rate and the pattern of shoaling in Savannah Harbor have changed drastically as the channel depth has been progressively increased, but the shoaling pattern has changed much more drastically than the shoaling rate. From data compiled by the U.S. Army Engineer District, Savannah, and especially by Ralph F. Rhodes, special assistant to the district engineer until his retirement in 1949, it has been possible to determine the approximate shoaling patterns and the average annual shoaling rate for the following four channel conditions which are shown in figure 4: (1) 30 ft. deep at mean low water from deep water in the ocean to station 190, thence 26 ft. deep to the Seaboard Airline Railway bridge, thence 21 ft. deep to station 94; (2) 30 ft. deep at mean low water from deep water in the ocean to station 190, thence 26 ft. deep to station 94, thence 21 ft. deep to Creosote Wharf; (3) 30 ft. deep at mean low water from deep water in the ocean to the Seaboard Airline Railway Bridge, thence 26 ft. deep to Creosote Wharf; and (4) 36 ft. deep at mean low water from deep water in the ocean to station 190, thence 34 ft. deep to station 94, thence 30 ft. deep to station 82.

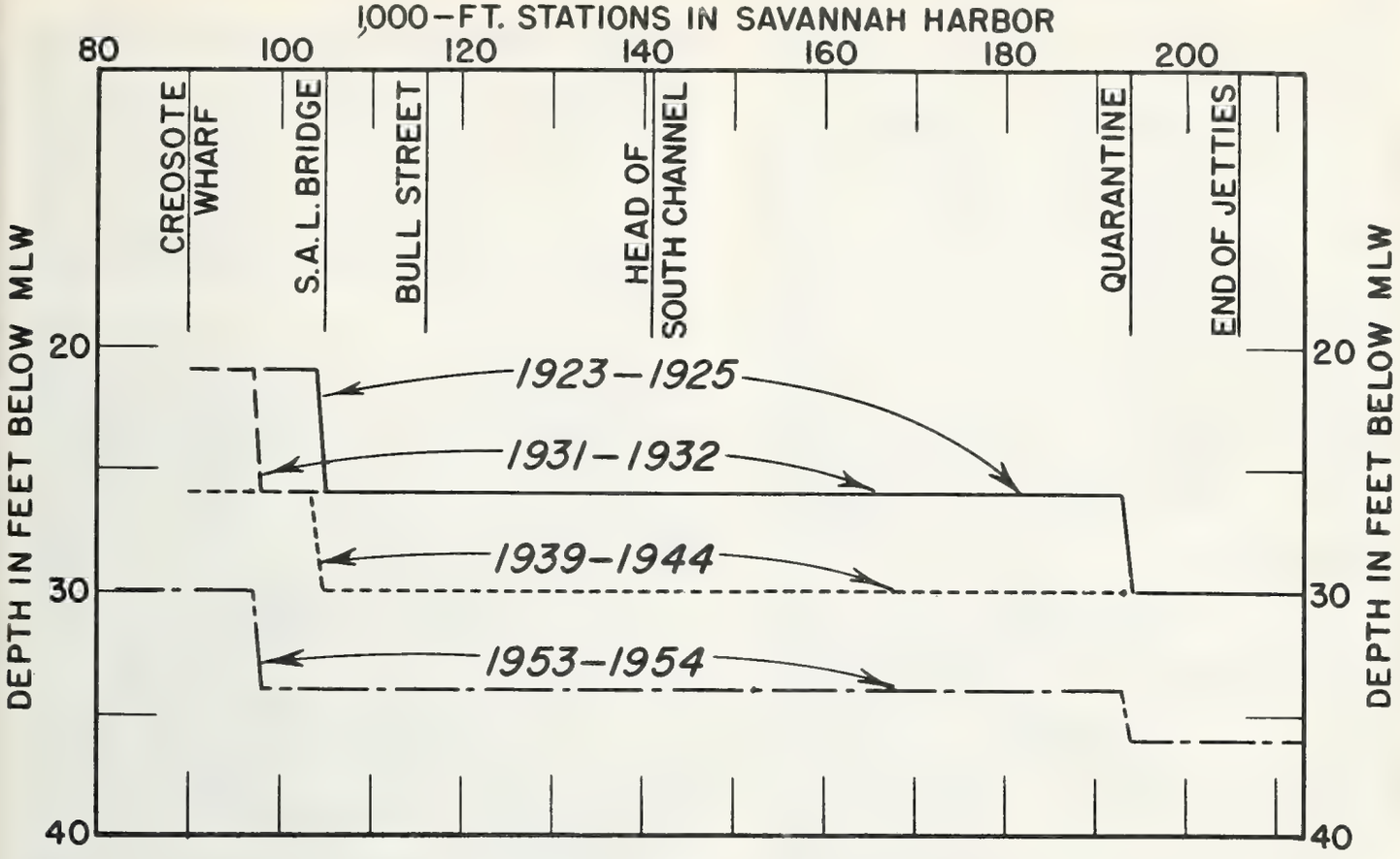

FIGURE 4. — Past and present channel depths.

For purposes of the analysis described below, data obtained for the period 1923–25 were used to represent the channel conditions described in (1) above, those for the period 1931–32 were used to represent the conditions described in (2), those for the period 1939–44 were used to represent the conditions described in (3), and those for the period 1953–54 were used to represent the conditions described in (4). These periods were selected because no changes in project channel dimensions were made immediately before or during any of these periods. The harbor was divided into the following three reaches: stations 107 to 134, representing approximately the upstream one-third of the harbor; stations 134 to 166, representing approximately the central one-third; and stations 166 to 205, representing approximately the downstream one-third. The annual average shoaling rate for each one-third of the harbor was determined for each of the four periods, as well as the annual average harborwide shoaling rate for each period.

The annual average shoaling rates for the upstream one-third (stations 107–134) of the harbor and the downstream one-third (stations 166–205) for each of the four channel conditions are plotted in figure 5. The points connected by the dashed line in figure 5 show the annual average harborwide shoaling rates, in millions of cubic yards, for each of the four periods. Examination of figure 5 indicates that the annual average shoaling rate for the lower one-third of the harbor was about 2.1 for the first channel condition, 1.5 for the second, 1.2 for the third, and 0.2 for the fourth, which shows a steady and significant *decrease* in shoaling with each deepening and/or upstream extension of the project channel. For the upstream one-third, the annual average shoaling rate was about 0.1 for the first channel condition, 0.9 for the second, 2.0 for the third, and 4.6 for the fourth, which shows a steady and significant *increase* in shoaling with each deepening and/or upstream extension of the project channel.

The annual average harborwide shoaling rate in millions of cubic yards was 2.8 for the first channel condition, 4.3 for the second, 6.2 for the third, and 7.2 for the fourth. Thus, the change from the first condition to the second increased shoaling by about 1.4 million cu. yd. per year, and the change from the second condition to the third caused a further increase of about 2 million cu. yd. per year. However, the change from the third condition to the fourth,

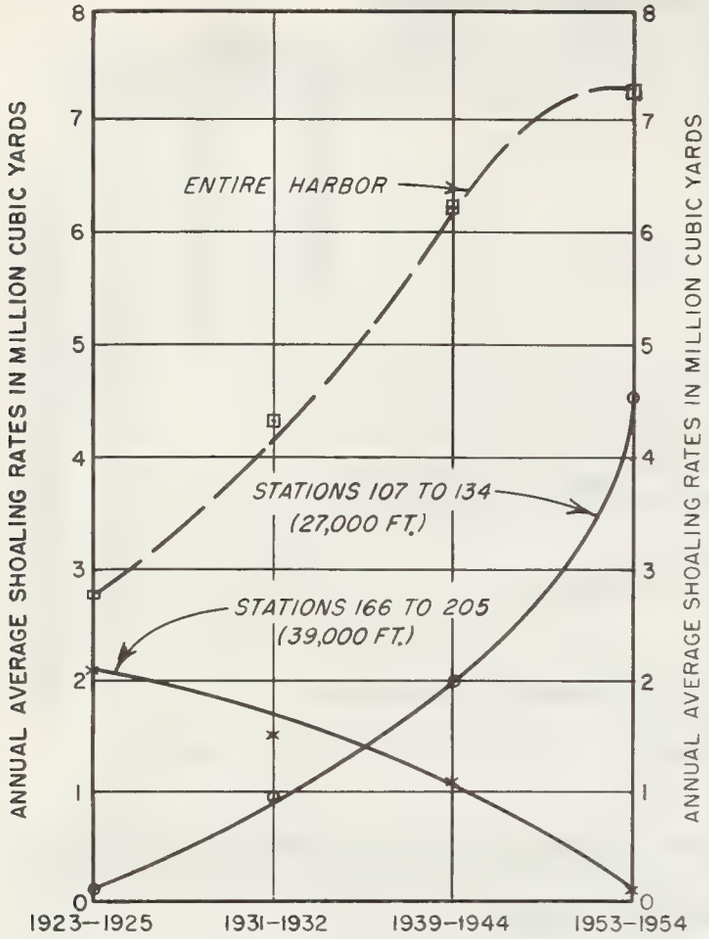

FIGURE 5. — Past and present shoaling characteristics of Savannah Harbor.

which was by far the most drastic physical change and the one that caused the maximum change in shoaling distribution, increased total shoaling by only 1 million cu. yd. per year. This trend indicates that by the time the third channel condition had been attained, density effects in the harbor had developed to the point that essentially all of the potential shoaling material from upland sources was being trapped somewhere within the system. Therefore, when the channel was further deepened and extended to the fourth condition, the changed density current regimen effected a further upstream shift in the location of the major shoal area, but the harborwide shoaling rate increased by only some 16 percent.

In the center one-third of the harbor, between channel stations 134 and 166, the annual average shoaling rate in millions of cubic yards was about 0.6 for the first channel condition, it increased to about 1.8 for the second, it increased further to about 3 for the third, and then decreased to about 2.4 for the fourth. In comparison to the downstream one-third of the harbor, the annual shoaling rate for the center one-third was much lower for the first channel condition, approximately the same for the second, about two and one-half times as great for the third, and more than 10 times as great for the fourth. In comparison to the upstream one-third of the harbor, the annual average shoaling rate for the center one-third was six times as great for the first channel condition, twice as great for the second, one and one-half times as great for the third, but only about one-half as great for the fourth.

These trends also demonstrate quite clearly the upstream migration of the major shoal area as the project channel was progressively deepened or extended, or both. For the first channel condition, the downstream one-third of the harbor experienced by far the greatest shoaling rate; for the second condition, the center one-third shoaled at a slightly greater rate than the downstream one-third and a much greater rate than the upstream one-third; for the third condition, the center one-third shoaled at a slightly greater rate than the upstream one-third and at a much greater rate than the downstream one-third; and for the fourth channel condition, the upstream one-third shoaled at about double the rate of the center one-third and at more than 20 times the rate of the downstream one-third.

Although the harborwide annual average shoaling rate increased only about 16 percent when the 30- and 26-ft. channels of condition (3) were deepened to 36 and 34 ft. as in condition (4), a very critical situation was created with respect to the disposal of dredge spoil. Downstream from about the mouth of Back River, spoil areas of almost unlimited capacity are available or can be readily obtained; however, upstream from this location the available spoil areas are being filled rapidly and there are no additional areas that can be developed economically for spoil disposal purposes. For the third channel condition discussed above, about 68 percent of the annual average shoaling rate of 6.2 million cu. yd. occurred downstream from the mouth of Back River, or where spoil disposal areas were readily available; however, for the fourth channel condition, about 64 percent of the annual average shoaling rate of 7.2 million occurred upstream from this location where spoil disposal areas were limited. The increase of about 2.6 million cu. yd. per year in Front River is rapidly filling the remaining spoil disposal areas, and the development of a critical spoil disposal problem is imminent unless corrective measures are taken to reduce shoaling in the upstream one-third of the harbor.

During the course of a recently completed hydraulic model investigation of Savannah Harbor at the Waterways Experiment Station, an improvement plan designed to reduce shoaling

in the upstream one-third of Savannah Harbor was developed and tested thoroughly. The general features of this plan are shown in figure 6 and consist of the following: (1) a deep basin,

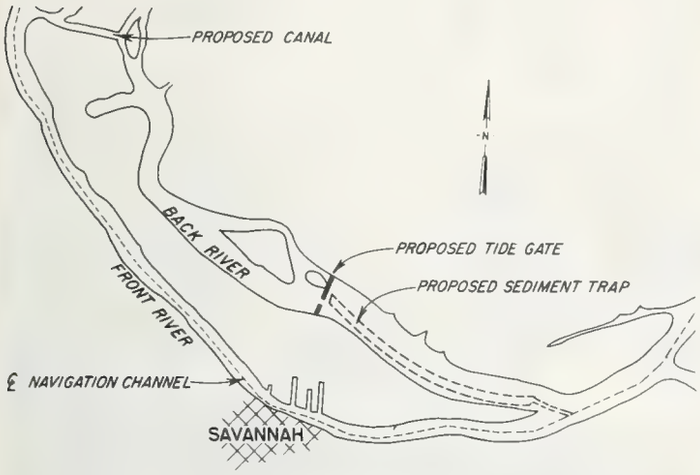

FIGURE 6. — Elements of sediment trap plan for Savannah Harbor.

or sediment trap, dredged in the downstream end of Back River and connected to the navigation channel by an access channel 200 ft. wide and 34 ft. deep at mean low water; (2) a dam equipped with one-way tide gates located in Back River just upstream from the sediment trap; and (3) a dredged channel, 200 ft. wide by 26 ft. deep at mean low water, connecting Back River and Middle River about 1 mile upstream from the confluence of Front and Middle Rivers.

The one-way tide gates in the Back River dam would allow the tidal prism of Back River to fill in the normal manner from the downstream end, thus permitting the inflowing currents to transport sediments from the navigation channel into the sediment trap. The gates would close when the Back River tidal prism upstream from the dam is filled, or when the water level on the two sides of the dam becomes equal. During the ebb current phase, that part of the Back River tidal prism upstream from the dam would flow through the dredge cut into Middle River and thence downstream through Front River, thus increasing the ebb current velocities in Front River to such extent that sediments would be moved downstream to the vicinity of the access channel to the sediment trap. Model tests have shown that this plan would be very effective in collecting in the sediment trap a major part of the sediments that now deposit in the upstream one-third of the harbor.

The above-described plan would not reduce the harborwide shoaling rate; in fact, model tests indicate that the harborwide shoaling rate would be increased slightly. However, because of the proximity of extensive spoil-disposal areas immediately north of the sediment trap and the fact that periodic removal of accumulated sediments from the trap to maintain its efficiency could be accomplished more economically than dredging from the channel proper, it appears that the plan would provide an acceptable and economic solution to the spoil-disposal problem. Furthermore, model tests indicate that, since most future maintenance dredging would be done from the sediment basin rather than from the channel, any given channel depth required for navigation could be maintained more effectively than by present dredging techniques.

In other words, the present practice of allowing the channel to shoal several feet above project depth and then dredging to about 2 ft. below project depth for advance maintenance would no longer be required except perhaps in isolated areas. Also, there is a strong probability that any remaining local shoal areas in Front River would merely require periodic agitation, rather than direct removal, since the net downstream flow in Front River with the plan installed should progressively move any sediments thus agitated downstream to the entrance to the sediment trap. Detailed studies are now in progress by the U.S. Army Engineer District, Savannah, to determine the technical and economic benefits of the plan in relation to the cost of its construction.

The Corps of Engineers makes detailed studies of every proposed change in navigation channel depth or extension in order to arrive at the best possible estimate of the effects of such changes on the quantity of maintenance dredging that will be required in the future. Until fairly recently, the exact location of major shoal areas was of little importance from an economic viewpoint, *since most of the recurring shoaling occurred downstream from major port areas, adjacent to lands not suitable for development,* where spoil areas were readily available. However, with increasing development of the lands adjacent to our tidal waterways for industrial and other purposes, and with the tendency for progressive upstream migration of the locations of major shoal areas because of changed density current regimens in the deeper channels, it is now of great economic importance to determine in advance the effects of a proposed deeper or longer navigation channel on the distribution of shoaling as well as on the quantity of shoaling. Even in those navigation channels where maintenance dredging operations are performed with hopper dredges rather than

pipeline dredges, every upstream migration of a major shoal area increases the haul distance to suitable dumping grounds and thus increases the unit cost of dredging.

In addition to the matter of changes in shoaling distribution discussed in this paper, the engineers of this era who are in charge of the development and maintenance of our tidal waterways are faced with many other problems that were of little or no concern to previous generations of waterway engineers. For example, the effects of a proposed change in navigation channel depth or length on the dispersion and flushing of industrial and municipal wastes must be known in advance from the standpoints of public health and water use. The effects of changed salinity and hydraulic regimens on the fish and wildlife resources of the area must be determined with maximum possible accuracy. Therefore, the modern waterway engineer should initiate his study of the effects of a proposed change in channel dimensions with a full knowledge of the effects of prior changes in the waterway involved, as well as of the effects of similar changes in other waterways of the same general type as that involved. He should be well versed in the principles of tidal hydraulics and should take advantage of every available tool, including hydraulic models when applicable and necessary, to insure that the new regimen of the waterway involved is determined with sufficient accuracy to satisfy the many and sometimes divergent interests involved. This challenge must be met if the continued development of our tidal waterways and the areas they serve is to proceed in a sound technical and economic manner.

SEDIMENT PROBLEMS OF RIVER DEVELOPMENTS IN DELTAIC REGIMES

[Paper No. 73]

By Chester A. Peyronnin, Jr., *hydraulic engineer, U.S. Army Engineer District, New Orleans*

Synopsis

Rivers carrying heavy sediment loads present many difficult problems for hydraulic engineers when attempts are made to develop these waterways for navigation or improve them for flood-control purposes. These problems are compounded when the river is actively building a delta system. Problems such as (1) shoaling in the navigation channels, (2) sediment control, (3) floodwater diversion, and (4) river migration must be resolved before full development of the river can be accomplished. These problems can be generalized and illustrated by typical examples of the Lower Mississippi River. This paper discusses each type, and gives some of the engineering considerations in the solution of each type of problem.

Introduction

The comprehensive development of a river for navigation and to control floods generally requires the resolution of problems concerning flood control, navigation, water supply, and recreational activities. In the case of a river that carries a heavy silt burden in a deltaic environment, these problems become very complex. Emergency relief floodways may become inoperable after only a few years of operation; navigation channels may become completely shoaled in a few weeks; and distributaries may be forced to migrate due to sediment deposition. Considerable research and analysis activities have been devoted to the understanding of the theoretical aspects of these problems, but most solutions must be based upon engineering judgment in the application of fundamental knowledge. This paper draws upon typical problems in the Lower Mississippi River Valley to illustrate possible solutions and the actual solutions employed to solve such problems.

General Description

The reach of the Mississippi River that will be used as an example extends from mile 130 above the Head of Passes to the Gulf of Mexico, 20 miles below the Head of Passes (fig. 1). Within this area are examples of navigational problems such as Southwest Pass, spillway and floodway siltation as typified by the Bonnet Carre spillway, migration and delta building considerations illustrated by the Cubits Gap delta, and delta destruction as shown by Isles Dernieres and the Timbalier Islands.

Navigational Problems

The most important problems in navigation projects is the maintenance of channel dimensions. Since the heavy silt load in the rivers under discussion will tend to deposit and form shoals under certain conditions, the maintenance of the channel is very difficult. The most critical location for the shoaling is generally at the mouth of the river where the silt-laden river water meets the heavier density seawater. The fresh water flows into the sea in the form of a

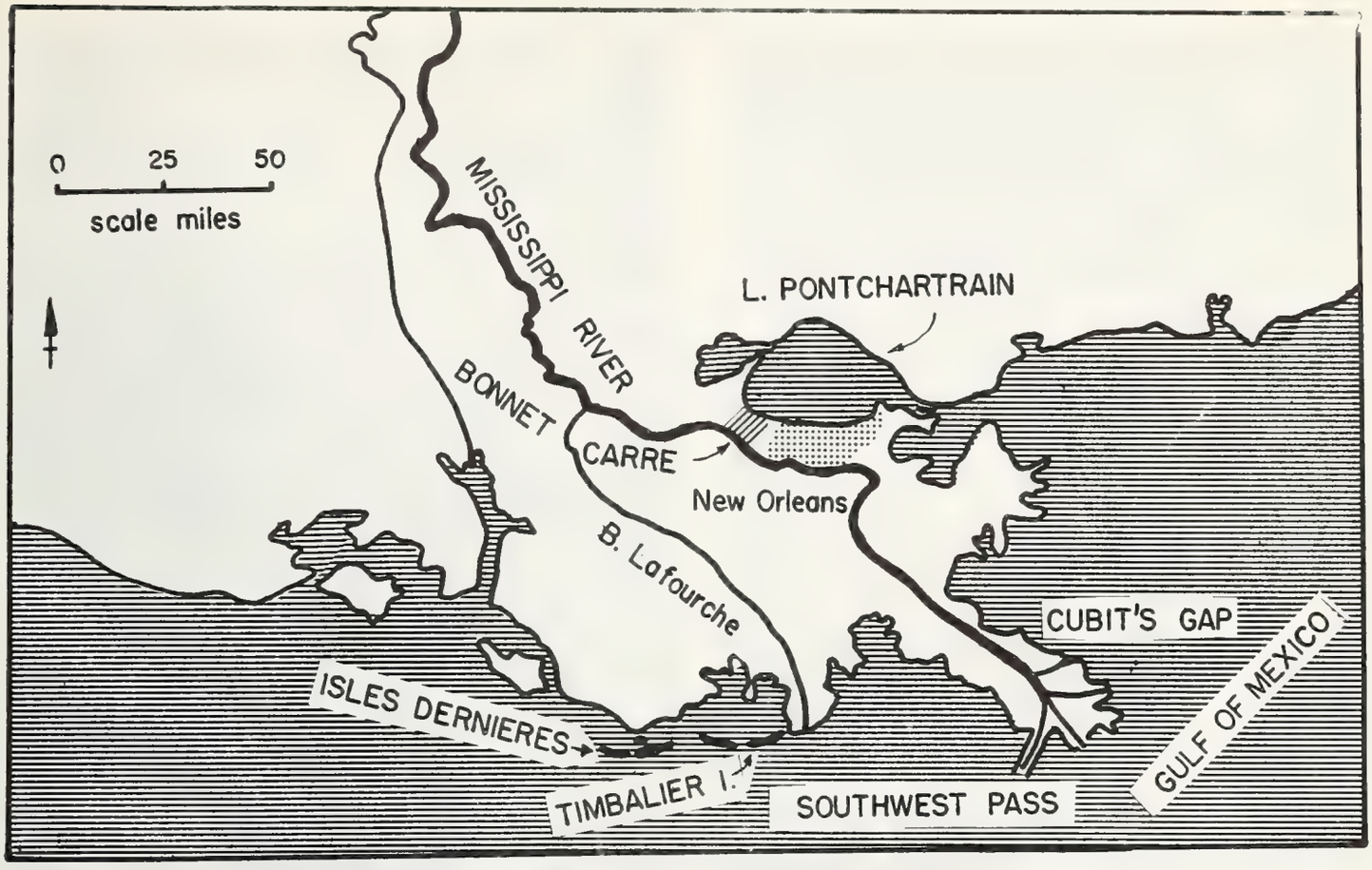

FIGURE 1. — Lower Mississippi River.

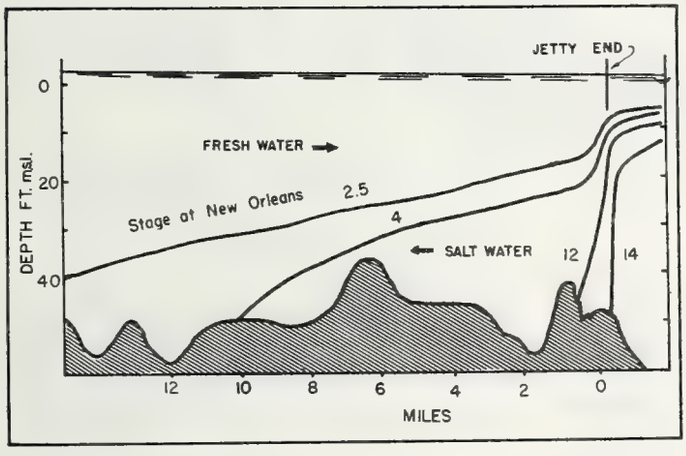

FIGURE 2. — Longitudinal profile of lower Mississippi River.

hypopycnal jet, with the lighter river water flowing over the denser seawater. An interface will be formed between the fluids, thus giving a characteristic wedge shape. This interface will be poorly defined because of mixing due to diffusion currents, but may be defined for study purposes as a line with a salinity expressed as 5,000 p.p.m. chloride ion. In rivers that have been improved for navigation, the deep channel will allow this wedge to move into the river at low stages. Salt water will flow upstream near the bottom while fresh water flows out near the surface. Any material moving near the bottom will be stopped by this wedge because of the local velocity reversal. At high rates of riverflow, this wedge will move out to the mouth, and since bed material will generally be moving at this river stage, a considerable amount of material will be stopped. The result will be a heavy shoal formation near the mouth or in the bar channel (fig. 2). Material that passes over this bar may be returned to the area by the upstream flow unless there are strong littoral currents to move it away.

The problem of heavy shoals in a bar channel may be solved in two ways. One way is to orient the main river channel so that the bedload and suspended material approach the wedge at an oblique angle and they will proceed into and over the wedge. Material carried over the wedge will be in a better position to be swept away, and material deposition at the toe of the wedge will be minimized. This method is shown in figure 3,*A*. An alternate method, which is somewhat of a departure from usual methods, is to allow the material to be deposited normally in the river channel while

navigation is carried on in an alternate channel (fig. 3,*B*). This channel must be positioned so that the salt wedge acts as a natural side to the river, thus stopping any material from moving into this pass. Some bottom flushing will be obtained by the scour action of the salt water. Model tests of both of these methods show benefits with respect to minimizing dredging in the order of 50 to 60 percent reduction over that of the previous alinement. The first method has been employed in the redesign of the Southwest Pass jetty channel and has shown good results in the actual condition. The second method showed model results equally as good as the first, but at a greater cost in this application.

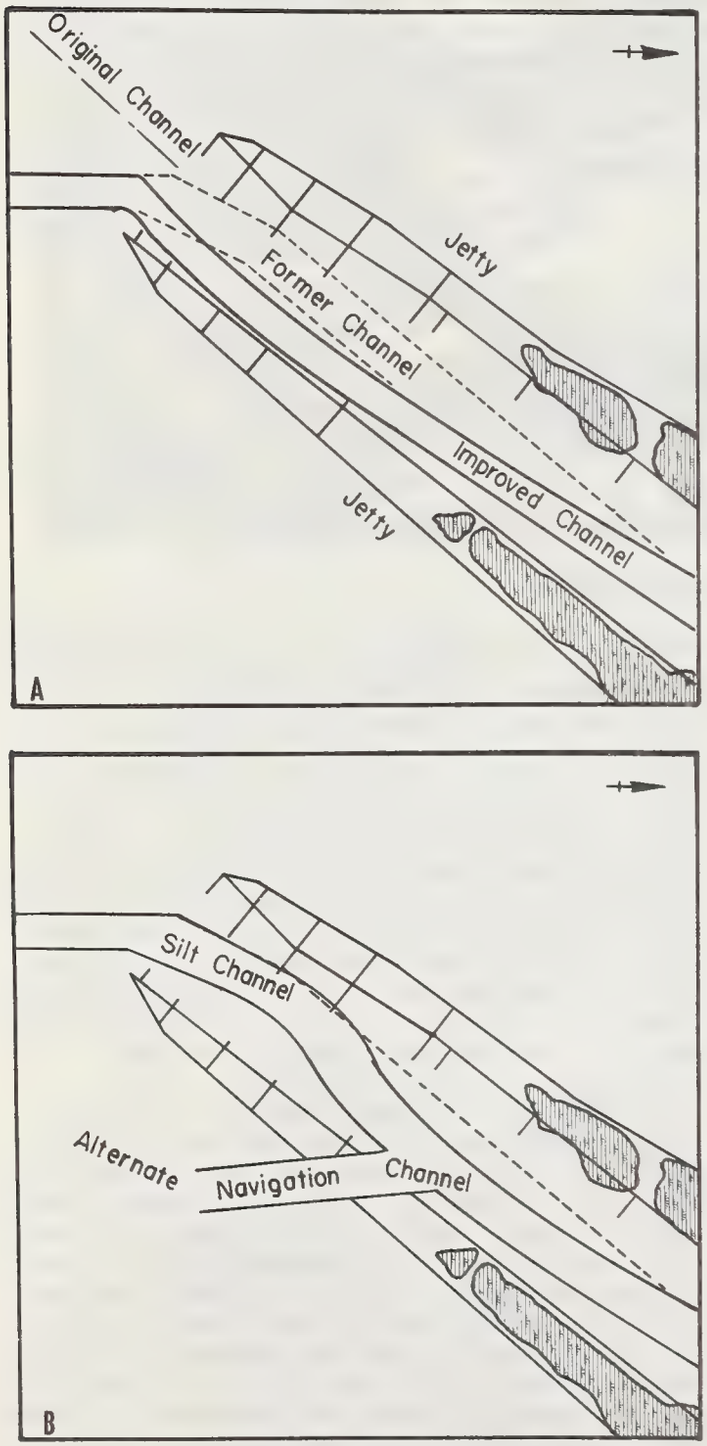

FIGURE 3. — Southwest Pass channel showing (*A*) improved alinement; and (*B*) alternate channel.

Another major problem of sedimentation is that caused by deposition under wharves or piers in the river. Material is trapped under these structures by piling bracing or by debris caught on the piles or deposited by receding high water. When the channel adjacent to the wharf is redredged to maintain the required depth, the material will slide and cause damage to the structure. In addition to slope slides, rotational failures have occurred due to the increased overburden as shown in figure 4. This problem can be solved by revising the channel maintenance program to insure that the sedimentary material is removed from under the structure. Routine maintenance can be improved to include removal of debris from under the structure at frequent intervals.

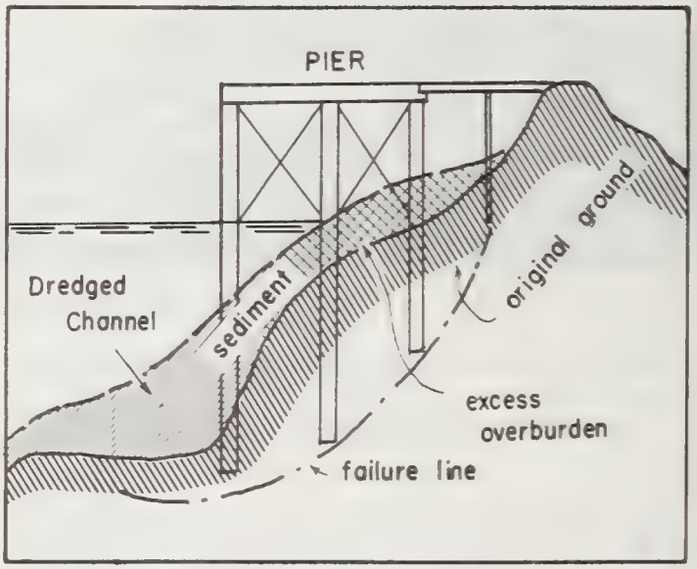

FIGURE 4. — Pier siltation.

Flood-Control Problems

In the natural state, a river will pass a flood by enlarging its distributaries, opening new ones, or flowing into a larger flood plain. In a highly developed river system, none of these can be allowed to happen. It is therefore necessary to provide some temporary system for increasing distributary flow. Spillways with floodways to the sea are commonly used for this purpose. There are three major sedimentation problems in such a system that may cause difficulty. The first is the repeated annual flood for

which the spillway is not opened. This will allow deposits in the forebay to build up and decrease the entrance efficiency to the control structure. The second is the deposition of sediment in the floodway, which will eventually reduce capacity after several operations. The third is the sedimentation in the outlet waters, which may decrease flow efficiency but which may pose more difficult problems with respect to the ecology of the area.

Bonnet Carre Spillway illustrates these problems rather well. It was completed in 1931 and has been operated in 1937, 1945, and 1950. Pertinent data of the River are as follows:

Drainage basin: 1,245,000 square miles.
Flows at New Orleans: Low, 50,000-100,000 c.f.s.; maximum, at project conditions, 1,250,000 c.f.s.
Stages at New Orleans (Carrollton): Flood stage, 17 ft. m.s.l.; maximum at project conditions, 20 ft. m.s.l.
Sediment sizes: General maximum — 1 mm. (0 phi units) in bed; 0.1 mm. (3.33 phi units) in suspension. Median — 0.025 mm. (5.22 phi units).
Dredging required in Southwest Pass (for previous 35-ft. project depth): Approximately 2¼ million cu. yd. mean annual; 6 million annual maximum.
Salinity of river: 30 p.p.m. chloride ion.
Salinity of Gulf of Mexico: 18,000 p.p.m. chloride ion.
Project depth of Southwest Pass: 40 ft. with 2-ft. overdepth allowed.

Amounts of sediment for each operation of the spillway are shown in table 1. There are problems in the spillway forebay and floodway. There is no problem in Lake Pontchartrain, but the potential of such a problem can be illustrated by the outlet conditions.

In the forebay, the buildup of sediment between operations has caused a reduction in flow area into the structure. During the 1950 operation, it was necessary to attempt to restore capacity by remedial dredging during operation. Although this helped by opening the forebay, it hurt the overall problem by putting more material into the floodway, thereby probably silting it up more than a normal operation would have. Several years after the 1937 operation, a small levee was built to close off the forebay from the river, thus minimizing the annual deposition. This was degraded in 1944 before an expected high water and remained degraded until after the 1945 operation. It has been restored but some deterioration has taken place. Solution of this problem by using this removable levee seems to be well justified, because the levee will prevent the undesirable annual deposition and retain full forebay capacity for the time of operation (fig. 5).

TABLE 1.—*Discharge and sediment fill deposited for Bonnet Carre Spillway for the years operated*

(Design capacity, 250,000 c.f.s.)

Operations	28 Jan. to 16 Mar. 1937	23 Mar. to 18 May 1945	10 Feb. to 19 Mar. 1950
Maximum discharge, c.f.s.	211,000	318,000	223,000
Average discharge, c.f.s.	131,000	223,887	156,217
Total discharge, acre-feet	12,400,000	24,500,000	10,900,000
Fill deposited, weir to Airline Highway:			
Volume, cu. yd.	7,200,000	5,000,000	2,800,000
Percent of total	70	40	56
Fill deposited, Airline Highway to Illinois Central Railroad:			
Volume, cu. yd.	3,000,000	7,500,000	2,200,000
Percent of total	30	60	44
Fill deposited in Lake Pontchartrain (approx.):			
Cu. yd.	2,230,000	17,500,000	7,400,000
Acre-feet	2,000	10,800	4,600

The floodway proper of such a structure will experience severe deposition during the operation period. In addition to the normal mechanics of deposition there is the problem of deposition due to interference in the flow by trees and brush. If the spillway can be kept clear of undergrowth and large trees, the flow line can be improved and the deposition decreased somewhat. Once the floodway capacity has been reduced by filling, the only solution is to dredge a channel. There are many possibilities in this restoration process, but the most economical seems to be the use of a single channel through the area because the spoil can be placed in the remaining part of the floodway at minimum costs. It would probably be necessary to clear this channel after every operation.

In the Bonnet Carre floodway, the pattern of deposition has been influenced to a great extent by flow conditions in the floodway. In 1937, the area from the structure to the Airline Highway had not been cleared, and so a large percentage of material settled out in this area. After the operation, the land was cleared and grubbed, and in 1945 and 1950, larger percentages of material were deposited in the section between the Airline Highway and the lake. Because of this deposition in the floodway, the 1950 flow line was elevated approximately 1 ft. above the 1945 flow line near the structure, and lesser amounts near the outlet. Because of the loss of capacity, the spillway can probably pass the design flow only once more before remedial measures are necessary.

The outfall sump of a spillway must receive all of the excess sediment from the floodway. The effect of such sediment may have two aspects, hydrological and biological. The sediment is deposited by two processes; the first by decreased velocity and sediment fallout, and the second by fresh-saline water interaction. This

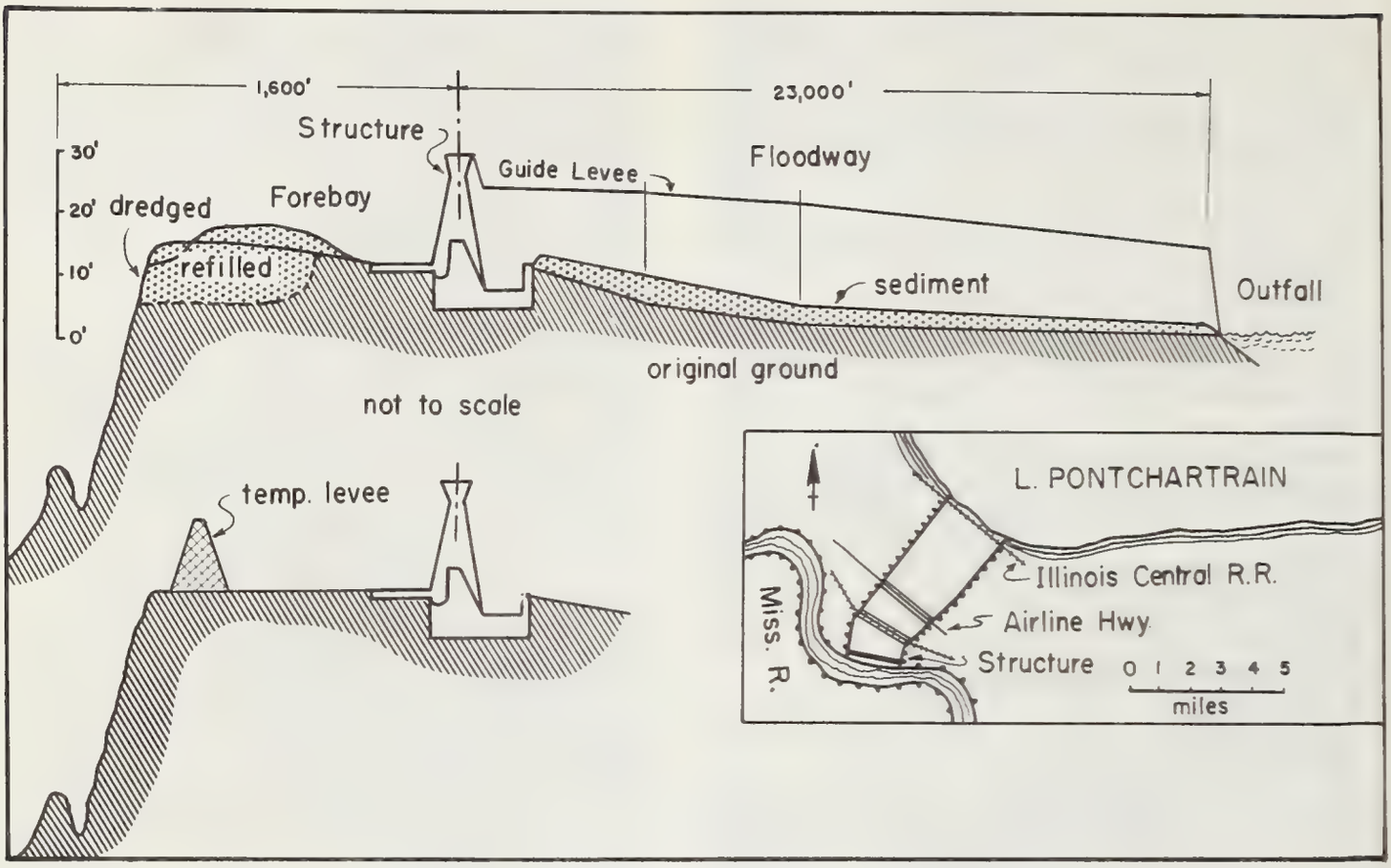

FIGURE 5. — Bonnet Carre spillway.

interaction results in flocculation which removes most of the suspended sediment. The remaining fine sediment will tend to increase the turbidity, thus giving the appearance of dirty water, but will not produce significant sedimentation because it will remain in suspension.

The quantity of sediment deposited in Lake Pontchartrain by each operation of Bonnet Carre is given in table 1, for a total of 17,400 acre-ft. All of this has been deposited within an area of 25 square miles adjacent to the floodway. At an operation frequency of approximately once in 10 years, this would amount to a 1-ft. decrease in lake depth over its 410,000 acres in a period of 825 years at present conditions. This is probably less sediment than the amount deposited by natural depositional processes active in the lake and less than the amount removed from the lake for various projects in the area that have used hydraulic fill from the lake. The effect upon the hydrology of the lake is thus relatively negligible. If this sedimentation is important in other situations, the only solution in a lake is remedial dredging. The effect upon the biological process is beneficial because of the nutritional value of the nutrient salts in the sediment. Thus, in this spillway, the outfall problem is negligible, although in others it may be important.

Delta Growth

Deltaic coasts are those that have been built by a depositional process in which the transport capacity of some parent stream carrying material into an area exceeds that of the local area sea currents carrying it away. The material forms a delta whose shape is controlled by the regime of the area in which it is deposited. Deltas that form in shallow saline water from a freshwater stream tend to form fan-shaped fronts with braided distributaries migrating through the delta. These distributaries are continually bifurcating and migrating as they silt up or are diverted by natural phenomena. Such [illegible] formation of the Mississippi River (fig. 6). Deltas that build in deeper water tend to advance in longer, narrower formations as illustrated by the growth at Southwest Pass of the Mississippi River (fig. 7). Since the delta is essentially submerged, it is outlined by the advance of the 35-ft. contour. The fan-shaped delta builds useful marshland while the finger-style delta builds underwater shoals of little contemporary usefulness.

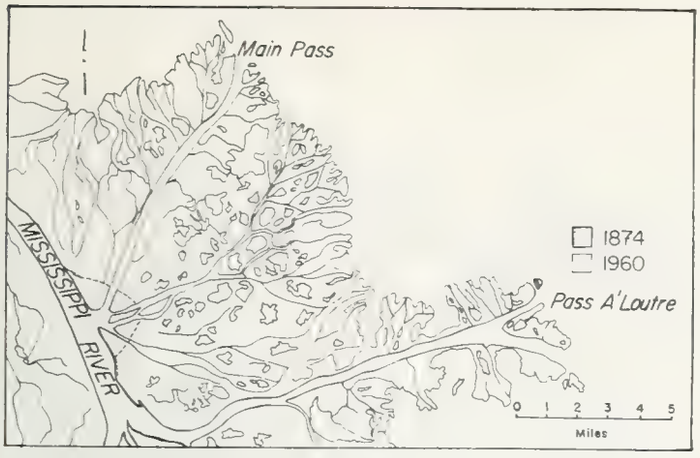

FIGURE 6. — Cubits Gap delta.

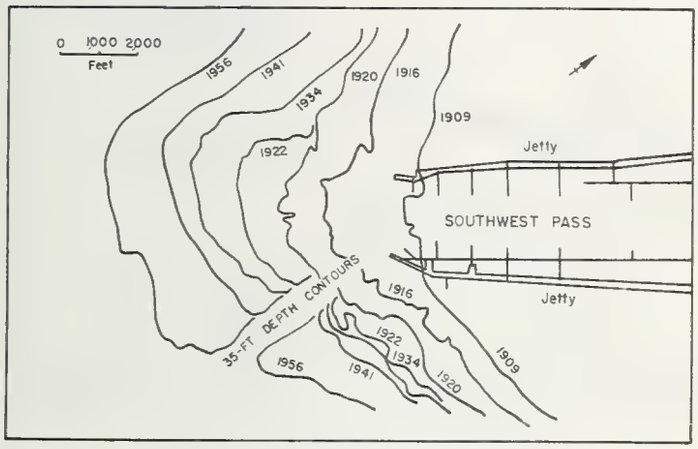

FIGURE 7. — Southwest Pass delta.

Any river work that tends to alter the natural sedimentary activity of the river will materially affect the delta. Navigation works generally require deep channels to offshore depths, which may be too deep for useful delta building. Flood improvements may require the closing of a distributary which will then rob an area of its source of supply. Power or flood-control dams may so obstruct the sediment load as to prevent an adequate amount of material from entering the delta and thus cause an imbalance in the equilibrium.

The extensive and detailed experimentation work at Southwest Pass has resulted in the improvement of flow conditions such that depositional effects of the salt wedge are minimized and the sediment load carried efficiently out to sea. Because of the great depths required for navigation, the bulk of the sediment is deposited in deep water and does not assist in the building of a useful delta (fig. 7). In order to assist in maintaining the integrity of the land along Southwest Pass, several controlled outlets to the Gulf are maintained and allowed to pass sediment to the bay area adjacent to the pass. Other than this small amount, all other sediment is wasted. This can be contrasted with the essentially natural channel forming the Cubits Gap delta.

Delta Destruction

Once a delta is abandoned because of migration of the parent stream, the erosional and subsidence effects, which have been present, become dominant and the delta is slowly destroyed. From an economic standpoint, it is desirable to maintain these deltas because this land, while generally not inhabited or developed, is the location of important interests such as fur trapping, and wildlife conservation. If the distal fringes of these deltas can be protected, the land will continue to be useful for extended periods because vegetation growth and decay will continue to offset the subsidence to a great extent.

These fringes are subject to two types of attack. The first type of attack is that of net subsidence which sets in once the active depositional process ceases. Subsidence has been in effect all during the active delta period with new material replacing that which subsided. Once inactive, the delta front will continue to subside. The fringe beaches will not subside as rapidly, but will form offshore islands backed by shallow bays. These islands will act as protective barriers against wave attack on the mainland, but in time the islands will yield to subsidence and erosion and the attack will continue on the mainland.

The second type of attack is that which is commonly associated with sea processes; that is, wave action and littoral current movement of material. In this attack the waves will work a sorting process, moving the fine material off the beach and into the nearshore bottom where the littoral currents can move it away. If this process can continue to equilibrium, a relatively stable beach will be formed on the delta front. However, this beach will generally be attacked by higher waves during storms and the stability upset, thus beginning the process again.

Timbalier Islands

The Timbalier Islands probably began as spits off of the mouth of Bayou Lafourche while it was an active distributary of the Mississippi River during both its deltaic growth in the Lafourche Delta and its later activity in the modern delta. Material was continually fed in small quantities to the mainland and islands until 1904, when Bayou Lafourche was closed off from the Mississippi River at its head for flood-control purposes. Once this source of sediment was cut off, the process of erosion became dominant. In the last 50 years, there has been a retreat of the mouth of the bayou at Belle Pass at a rate of 120 ft. per year (fig. 8).

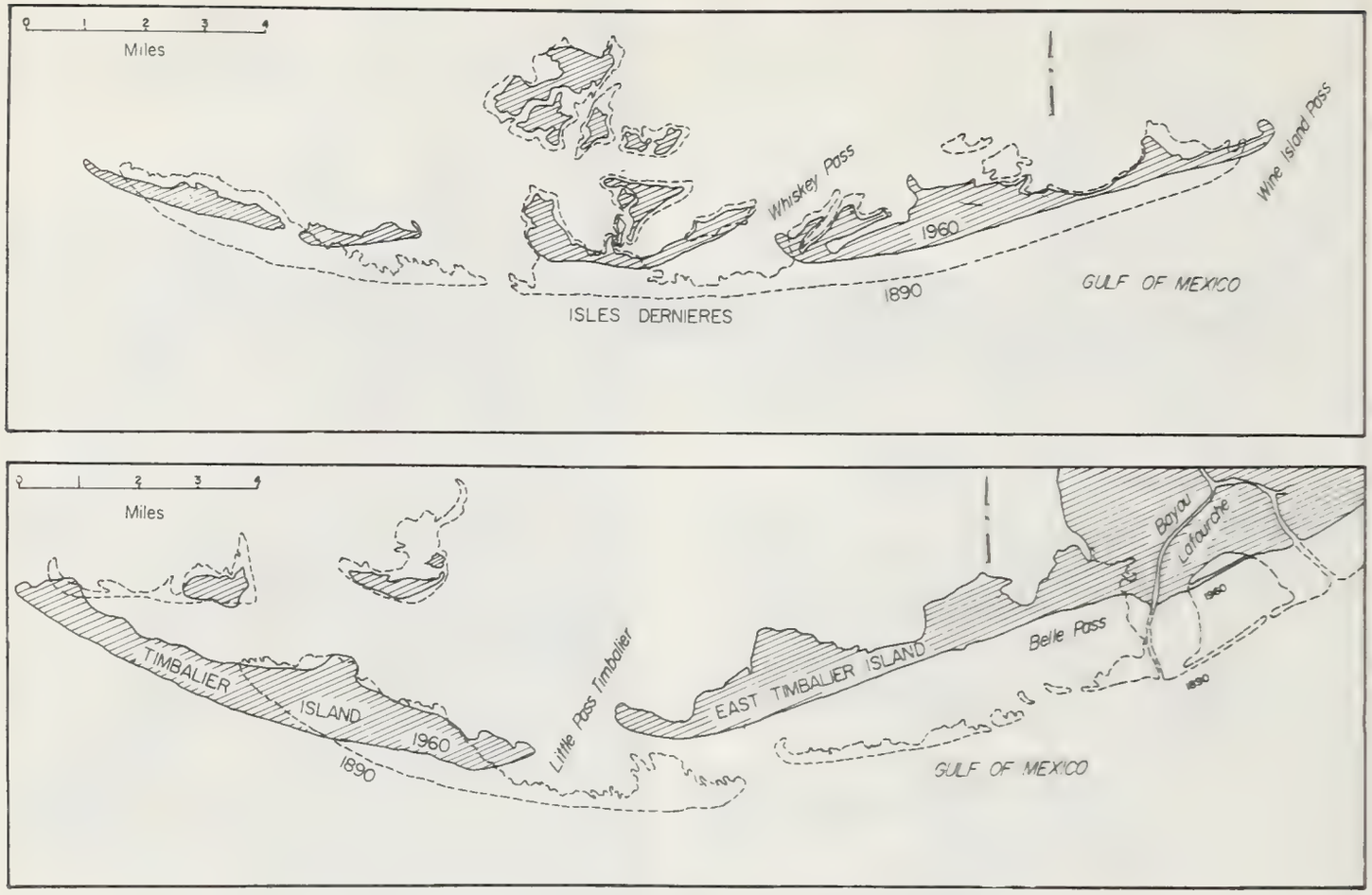

FIGURE 8. — Timbalier Islands and Isles Dernieres.

The littoral currents in this area are westerly and wave action from the southeast and south has a westerly longshore component, so that material which is eroded from this headland drifts westerly along the island chain. The chain is apparently building a continuous strip, but is experiencing a net loss of material in the planing action. The east end of East Timbalier Island has been retreating at a rate of approximately 100 ft. per year, and the western end at a rate of 75 ft. per year. Timbalier is retreating at a rate of only 50 ft. per year at its eastern end and is advancing on its western end as a result of the planing action. Since no new material is being added to this general mainland area, it will continue to erode and the islands will continue to erode with it. As more of the coast is attacked the rate will decrease, but eventually the islands will be pushed back to the main marsh area which, in turn, will be subject to attack.

For this regime, there is no solution for preservation. Seawalls, bulkheads, or breakwaters are too expensive for the area. Artificial nourishment is also out of the question because of a lack of available material for nourishment. Maximum sand size on the beach is 0.5 mm. and median size is 0.175 mm. No sand equal to or better than this is available in the general region for replenishment. There is some sand up in the mainland along Bayou Lafourche, but not in the quantity required, and it would be inadvisable to rob the immediate mainland that is being protected. Some fresh water is now being diverted into Bayou Lafourche from the Mississippi River by a pumping station, which began operation in 1955, but not in sufficient quantities to provide any significant amount of material.

Isles Dernieres

The Isles Dernieres are remnants of the distal extremities of the old Teche Delta of the Mississippi River, which was active between 3,000 to 4,000 years ago. While the delta was active, there was a general fan-shaped alluvial delta advance into the sea. When the river migrated away, the delta front remained and was reworked into sandy beaches fringing the delta. Subsidence of the land behind these beaches caused the beaches to emerge as islands separated from the mainland by bays.

Since no new material is moving into this

area, the islands are being reworked and are succumbing to the action of wave attack (fig. 8). At the present time, the central part of these islands is receding at a rate of approximately 40 ft. per year. The eastern end is receding at a lesser rate of 24 ft. per year and the western end at the rate of 12 ft. per year. The ends are advancing slightly, due to the littoral transport of material from the center section to the ends. The littoral current off the islands generally flows from east to west with the exception of a local reversal at the eastern end of the islands. Waves from the southeast, south, and southwest refract almost perpendicular to the shore so that there is only limited lateral motion of the beach material. There is a general reduction in shore area and both shore and nearshore bottom volume. The indications are that these islands are being planed away and will gradually disappear. When the islands are gone, there will be a direct attack upon the mainland, which is predominantly a salty marsh. This mainland will again develop a frontal beach due to the removal of the fine material, but this in turn will be attacked and continuously destroyed by wave and storm attack.

The beach material on these islands is rather fine, having a maximum size of 0.5 mm. and a median size of 0.15 mm. There are no suitable sources of nourishment material available near the islands, so that artificial nourishment cannot be employed. Groins would not help, because there is very little movement along the beach. An offshore breakwater or seawall would be too expensive and would require costly maintenance because of the general subsidence of the area and would be economically unjustified because of the lack of development on the islands or adjacent mainland.

Solutions

The standard solutions in beach-erosion control are the use of structures such as groins, seawalls, or breakwaters; the replenishment of the beach front; and the stockpiling of material to be placed by littoral processes. These are generally used singularly or in combination, depending upon the particular problem. All such solutions require the expenditure of comparatively large sums of money and, when measured against the benefits, yield a low benefit-cost ratio because of the expanse of area to be protected and the sparseness of habitation.

The best technical solution for such an area lies in the reestablishment of the inflow of sediment-laden water into it. This means that the parent stream, or branches of it, must be returned to provide natural nourishment to the area. This would seem to be an almost impossible solution if present navigation and flood-control concepts are considered. Newer thinking has presented some interesting possibilities that would make this concept worthy of consideration. For instance, with regard to the example, the Mississippi River has long been an important navigable waterway and a vast sum has been expended to develop and maintain the mouth of it as such. To the east of it, however, there is a tidewater channel being constructed from New Orleans to the Gulf of Mexico called the Mississippi River-Gulf Outlet, which will allow seagoing vessels to reach the city and enter the Mississippi River through a lock that may be constructed in the future if justifiable. This has many advantages over the river since it is not affected by flooding cycles or serious siltation problems. It thus appears feasible to construct ship channels to handle commerce to inland ports without utilizing major waterways. Below a particular point on a stream, it is possible to allow the stream to return to a modified natural meandering with many distributaries. These smaller more numerous openings would pass floods on wider flood plains and build a more useful delta.

It is not proposed to allow completely unrestricted action, since this would present many additional problems. The only practical solution to the erosion of deltas lies in the long-range preplanned control of distributaries, utilizing control structures and revetment of directional control of many of the distributaries to prevent undesirable migration. This system has been used to a limited extent in certain locations, such as along Southwest Pass of the Mississippi River, where an attempt has been made to maintain the status quo of the land adjacent to the navigable pass by controlling the several side outlets so that there is no change in the flow and sediment passing out through them but sediment is carried out so as to fill the land.

In deltaic areas that have not developed commercially or industrially, the plan would be easily implemented. Any flood-control or navigation work on rivers could be planned to include the preservation of the delta system below a certain point in the main stream.

Conclusions

In river systems, there are many problems involving sedimentation from various sources. These are increased by river developments and, generally, the more extensive the development, the more complex are the problems. All of this can be solved or minimized by good engineering design after careful analysis of the problems.

SEDIMENT DISCHARGE TO THE COAST AS RELATED TO SHORE PROCESSES

[Paper No. 74]

By GEORGE M. WATTS, *hydraulic engineer, Beach Erosion Board, U.S. Army Corps of Engineers, Washington, D.C.*

Synopsis

The supply of sediments to the shore by stream and river discharge is a very important factor in the stability of the shoreline. When this supply is altered, littoral forces will accordingly try to adjust the shore zone. The development of works on upland drainage basins for flood control and other purposes generally reduces the sediment supply to the coast and the resulting effect to the shore zone will be that of erosion. Unless an appropriate plan of shore protection is developed and constructed, valuable coastal property may be lost.

This paper discusses two coastal areas — Ventura and northern Orange County, Calif. — where the supply of sand to the coast has been greatly reduced by basin development. In each case the effects on the shore zone are pointed out and the plan of improvement developed to solve the resulting shore erosion problem.

Introduction

Engineers have long recognized the importance of sediment transport to the coastal zone by river or streamflow and the general effect on this zone if the sediment supply is altered. As the development and control of drainage basins tributary to our coast progress, it is apparent that sediments supplied to the shore zones from this source are being reduced. Quantitative survey data indicate that general erosion prevails along our coast, but these and other data are not sufficiently definitive to evaluate what part of the erosion is due to a reduction of sediments supplied to the coast by stream and river flow.

The stability of a shore or beach zone depends, for the most part, on there being a balance between supply and loss of material in the littoral zone. If a reduction in supply occurs from a reduction of sediment supply by upland streamflow, the littoral forces (mainly, waves and wave-induced currents) will tend to erode the backshore to obtain materials to balance this reduction. On the other hand, the retention of sediments by upland development would appear to provide benefits related to the problem of sedimentation within navigation channels at coastal inlets. This may be true in certain instances, but it is generally not the case. The reduction of sediment supply to the coast increases the unfavorable condition of unbalance between applied littoral forces and material movement, and if the composition of the backshore is mainly fine-sized materials, the shore may recede a considerable distance. Under these conditions it may well be that the actual quantity of littoral drift moving along the shore and into an inlet channel will be substantially greater as compared to the condition where flow from upland streams provide a supply of suitably sized sand for nourishing the shoreline for some distance along the shores adjacent to the inlet.

The U.S. Army Corps of Engineers has a direct interest in this overall aspect. The chief of engineers is responsible for making studies and carrying out beach erosion and navigation projects, in accordance with appropriate Federal Laws and Acts pertaining to the Federal role in such functions.

Purpose

This paper presents cases where the supply of material to the coast by upland flow was greatly reduced, points out the resulting effects to the shore zone, and summarizes the plans developed to improve or stabilize the shore zone.

Presentation of Cases

Ventura, Calif.

General

The shore sector near the city of Ventura, known as Pierpoint Beach, is about 75 miles northwesterly of the city of Los Angeles. Sediments contributed to the coast in this area are from the Santa Clara and Ventura Rivers. Location of the area is shown on figure 1, the drainage basin of the rivers on figure 2. The dominant direction of littoral transport, i.e., the dominant direction of alongshore movement of littoral materials by the littoral forces, is southerly, but, due to variability in direction of wave attack, frequent transport of beach materials to the north does occur. The Santa Clara River delta advances seaward following periods of excessive flooding and tends to act as a barrier to littoral drift moving downcoast from the Ventura River mouth and the coast north thereof. The barrier effect extends to the Pierpont Beach area, and the advance or retreat of the beach in the Pierpont area has been greatly influenced by fluctuations of the Santa Clara River Delta.

Sedimentation and Other Data[1]

The mouth of the Santa Clara River is located

[1] Data from "Special Interim Report on Ventura Area," Beach Erosion Control Report, U.S. Army Engineer District, Los Angeles, Calif., August 1961; House Doc. 458, 87 Cong., 2d Sess.; House Doc. 29, 83d Cong., 1st Sess.

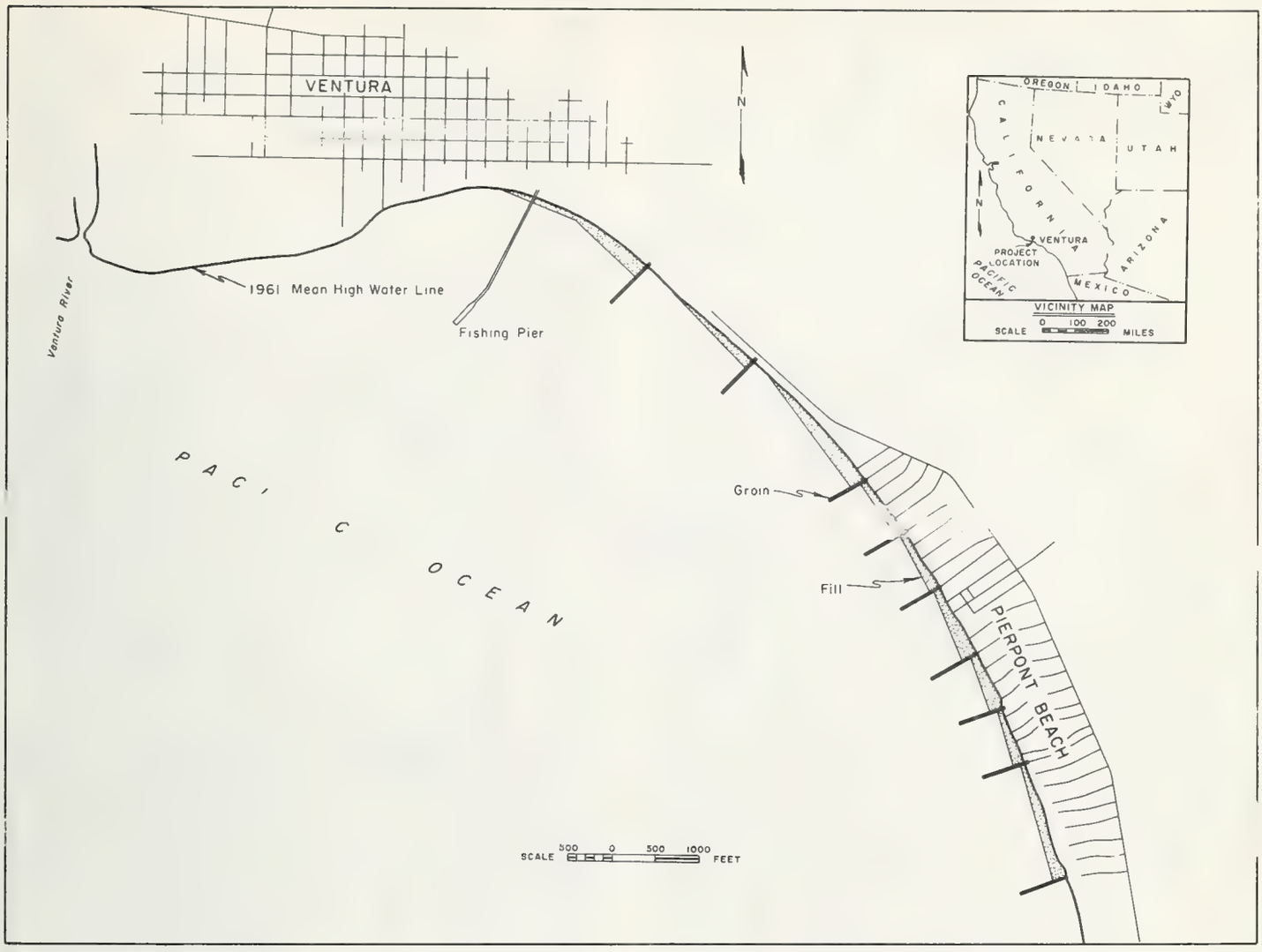

FIGURE 1. — Ventura area, California, general plan of improvement.

about 5 miles downcoast of Ventura. The drainage basin of this river comprises about 1,605 square miles, with about 90 percent of the drainage from the inland slopes of the western San Gabriel Mountains and the southwestern slopes of the eastern Santa Ynez Mountains. Slopes of the river in the mountains average about 160 feet per mile and about 15 feet per mile on the coastal plain. The basin is typical of this part of California in that it consists mostly of sand-producing sedimentary rocks. The estimated seasonal runoff for the river over a 40-year period (1893–1932) was 152,100 acre-feet. The estimated annual sedimentation rate (sediments that could reach the stream and be subjected to transportation and/or deposition therein) for the Santa Clara River is as follows:

Santa Clara River sedimentation data (1893-1932):

Drainage area	sq. mi...	1,605
Sand-producing part of drainage area	sq. mi...	960
Rate of sedimentation per square mile	cu. yd...	2,9[illegible]
Average annual sedimentation of sand	cu. yd...	[illegible],000

It was estimated that the total quantity of sediment reaching the coast each year was about 50 percent of the average annual sedimentation rate, or less than 1,400,000 cubic yards. Construction of the Santa Felicia Reservoir in the Piru tributary drainage area reduced the annual sand contribution by an estimated 340,000 cubic yards. The effect of other reservoirs constructed since 1948, along with the effects of a series of extremely dry years, has further reduced the sediment contribution to the coast, and the average since 1948 is now estimated at between 633,000 and 840,000 cubic yards per year. Thus, where there was 1.4 million cubic yards per year reaching the coast southeast of Ventura prior to 1948, there is now only about 50 percent of that amount. Though the Santa Clara River sediments mainly provide

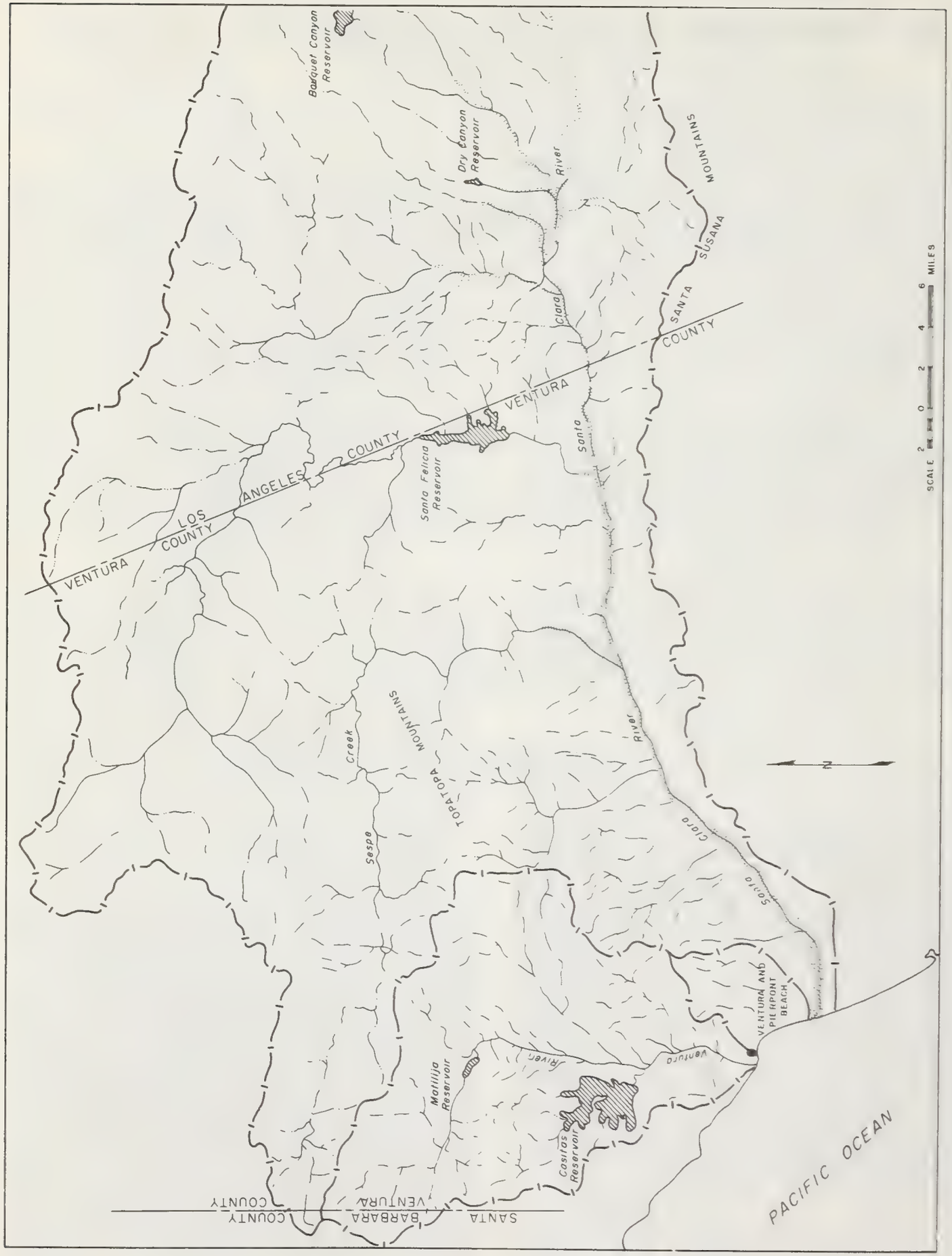

FIGURE 2. Drainage areas, Ventura and Santa Clara Rivers, Calif.

nourishment for the shores downcoast of the river mouth, some nourishment to the upcoast shores in the Ventura area occurs by reason of periodic reversal in direction of littoral transport, i.e., to the northwest. As mentioned, the seaward position of the delta greatly affects the stability of the Pierpont Beach sector due to the groin effect of the delta.

The mouth of the Ventura River is near the northerly limit of Pierpont Beach and the city of Ventura. The drainage basin of the Ventura River is 228 square miles, and it is enclosed on the north, east, and southeast by the Santa Clara River basin and bounded on the west and southwest by the Santa Ynez River basin. Gradients are, in general, comparable with those of the Santa Clara River. The annual sedimentation of sand for the Ventura River prior to 1948 was as follows:

Ventura River sedimentation data (before 1948):		
Drainage area	sq. mi...	228
Sand-producing part of drainage area	sq. mi...	149
Rate of sedimentation per square mile	cu. yd...	4,260
Average annual sedimentation of sand	cu. yd...	635,000

No data are available on the amount of sediment carried by the Ventura River to the shore, but, based on the data for the Santa Clara River, it was estimated that 50 percent reached the shore, or about 300,000 cubic yards per year. Due to construction of reservoirs on Matilijia and Coyote Creeks since 1948, it is estimated the sediment contribution to the ocean has been reduced by 50 percent.

The sediment yields to the coast for the Ventura and Santa Clara Rivers (located northwest and southeast, respectively, of Pierpont Beach) are summarized in table 1.

TABLE 1.—*Summarized sedimentation data for Santa Clara and Ventura Rivers*

Stream	Uncontrolled drainage area		Estimated contribution to littoral supply	
	Prior to 1948	After 1948	Prior to 1948	After 1948
	Sq. mi.	*Sq. mi.*	*Cu. yd.*	*Cu. yd.*
Ventura River	131	131	180,000	105,000
Matilijia Creek	56	.....	82,000	.....
Coyote Creek	41	.....	50,000	.....
Santa Clara River	845	845	680,000	395,000
Piru Creek	440	.....	335,000	.....
Sespe Creek	270	270	330,000	190,000
Santa Paula Creek	50	50	45,000	26,000
Total	1,833	1,296	1,702,000	716,000

Discussion

Thus with a gradual reduction of sediment supply to the coast due to upland development, there could be expected erosion of the coastal sector in the Ventura area. Reduction in sediment supply from the smaller sediment contributing river (Ventura), located northwest of Ventura, amounted to about 200,000 cubic yards per year. Since the dominant littoral transport is to the southeast, it was not surprising to find the annual erosion of the coast in the Ventura area amounted to a little less than this figure. Along the 2 miles of shore in the Pierpont Beach area the mean high water shoreline receded landward an average of 200 feet over the period 1948–61. Onshore installations, which previously had sufficient beach width fronting them to provide protection against severe wave attack, were gradually becoming vulnerable to damage by storm waves.

In evaluating plans of improvement for the problem area, it had to be assumed that the sediment contribution to the shore by the Ventura and Santa Clara Rivers would not increase in future years but, in fact, would probably be further reduced. Inland sources of suitable sand supply (by dry dryland haul) in this area are limited, thus auxiliary structures for the purpose of minimizing the annual losses of any material placed along the shore by mechanical means had to be considered. Considering all data and factors pertinent to the problem, a plan of protection was developed that involved construction of 9 groins and placement of 1,534,200 cubic yards of restoration fill along the Pierpont Beach sector. Required periodic nourishment for the restored beach was estimated to be 20,000 cubic yards per year. The recommended plan is shown on figure 1. Inland areas were located that had sand of suitable size characteristics and that contained sufficient volume to provide the initial fill for restoring the beach and future nourishment requirements.

The plan of protection is presently in the first phase of construction, which involves construction of three groins in the central and southerly sectors of the problem area and provides sand fill between the groins.

Northern Orange County, Calif.

General

The northwesterly limit of the Orange County shoreline is at the mouth of the San Gabriel River. The mouth of the Los Angeles River and the Los Angeles-Long Beach Harbor complex is about 4 miles upcoast from the San Gabriel River. The mouth of the Santa Ana River is located about 12 miles southeast of the San Gabriel River. Location of the area is shown on figure 3. These rivers are typical of southern California, as they transport large quantities of sediments to the coast during flood periods and relatively small quantities under normal flow. High flood conditions were experienced in 1938, which contributed large quantities of sand to the coast in the Los Angeles-northern

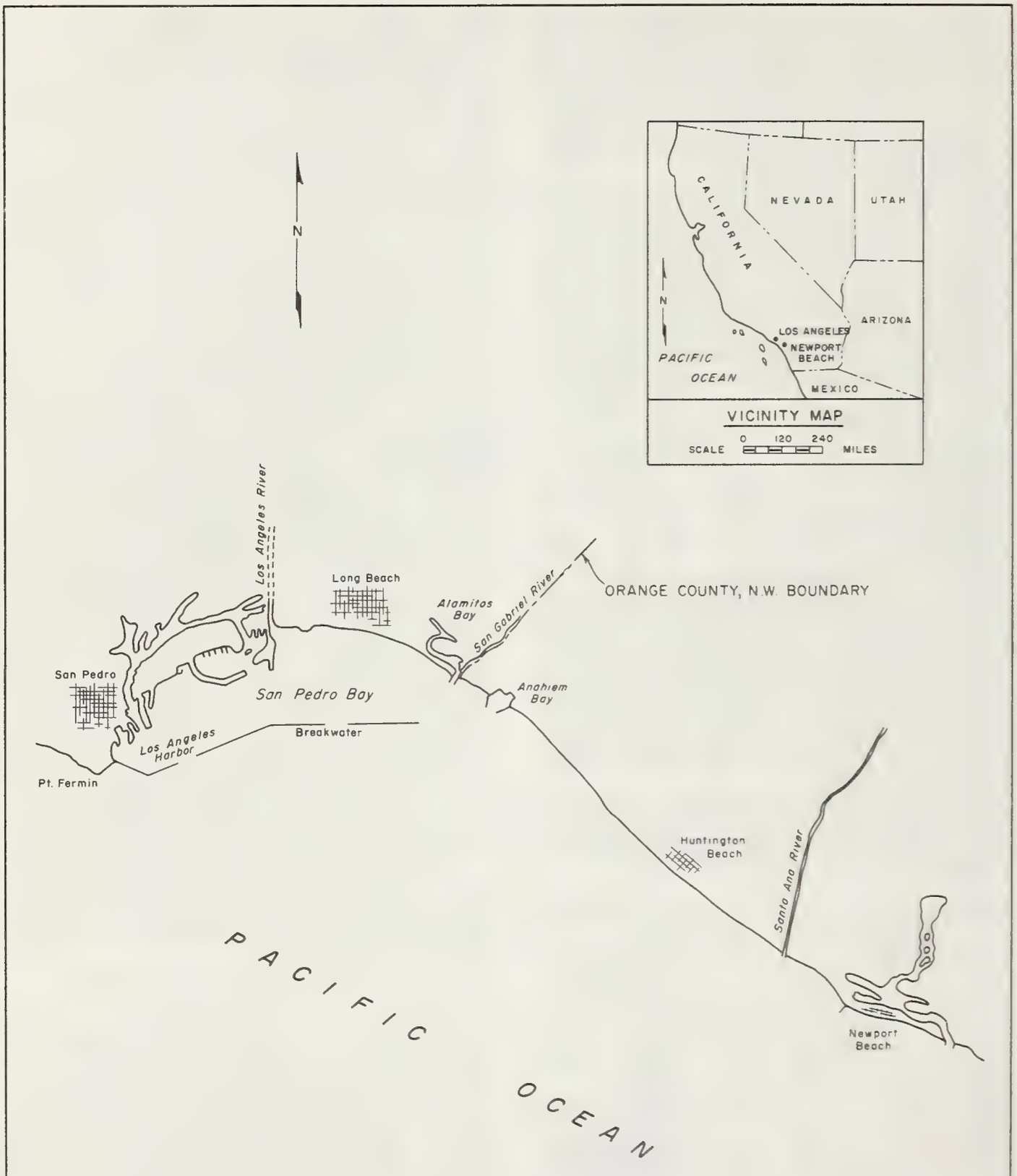

FIGURE 3
LOCALITY MAP

FIGURE 3. — Locality map, Los Angeles and Newport Beach.

Orange County areas. Since that time the upland has undergone such extensive development that should comparable flood conditions prevail, the sediment yield to the coast would undoubtedly be much less than that in 1938. This general aspect has been a factor in the shore erosion problems in the northerly reaches of Orange County, particularly between Newport Beach and Anaheim Bay. Several structures have been built along this shore sector that interrupt the littoral drift and affect shore stability; however, reduction of upland sediment supply has undoubtedly been one of the primary factors causing instability of this shore sector.

Sedimentation and Other Data [2] [3]

The Los Angeles River outlet works, located about 4 miles north of the Orange County line, are not actually within the shore erosion problem area. However, this river has had a very significant influence on the problem and should be considered before discussing the San Gabriel and Santa Ana Rivers and the problem area. Los Angeles-Long Beach has an interesting history of development [4] as does the ocean outlet of the Los Angeles River.[5] The mouth of the river at one time was located in the area that is now the heart of the harbor complex. Prior to the extensive harbor development the river must have discharged large quantities of sand to the shore. Although waves approaching this shore sector vary in direction and are influenced by offshore islands, the resultant littoral forces produced a dominant southeasterly movement of the littoral drift. The river sand served to nourish the shore for a considerable distance to the southeast.

The origin of the Los Angeles River is in the Santa Susanna and Santa Monica Mountains. Its drainage basin is shown on figure 4. It flows along the south side of the San Fernando Valley, around the easterly terminus of the Santa Monica Mountains, thence across about 22 miles of the Coastal Plain to Long Beach. The watershed of the river makes up an area of about 818 square miles. A channel, trapezoidal in cross section, was completed in 1923 in the Long Beach area, which served to divert the lower reaches of the river and establish the ocean outlet just east of the harbor.

By 1941 the offshore San Pedro-Long Beach breakwater extended southeasterly a sufficient distance to cause dampening of the wave energy reaching the shore in the river outlet area and reduced the effectiveness of the waves to dissipate the large shoal at the mouth of the river. Survey data covering the river mouth and adjacent shore for the period of 1923–38 showed the river discharge contributed over 500,000 cubic yards of material per year. During the 1940's large quantities of material were dredged from the channel through the delta, some being placed on a 4-mile length of beach to the east of the river outlet (about 6,000,000 cu. yd.) and some placed in the harbor area to create land for improvement of harbor facilities (about 8,200,000 cu. yd.). Since 1938 this river basin has been highly developed, including extensive urban and industrial development, check dams, and many channel stabilizing measures. At present the quantity of sediment arriving at the river mouth is a very small fraction of that supplied prior to 1940. The offshore breakwater and deepened channel at the mouth of the river greatly affect natural littoral transport of the river sediments to the shores east thereof; however, these factors are of little importance to alongshore material transport if the small quantity of sand now supplied by the river discharge is considered. It is apparent that substantial adjustments, principally erosion, could be expected for the shores southeasterly of Long Beach when the large upland supply of materials to the coast through the Los Angeles River was virtually eliminated. This problem for the shore sector from the Los Angeles River to the San Gabriel River has been minimized and, in fact, the area is fairly stable, due to the southeasterly extension of the Los Angeles-Long Beach offshore breakwater system and fill placed along the shore (i.e., the 6 million cubic yards mentioned above and recent material placement from a harbor development in Alamitos Bay).

As far as is known, there are no available data on the quantity of sediment discharge to the coast from the San Gabriel River. The drainage basin of this river is shown on figure 4. It has an area of about 700 square miles. The development of works in the upper reaches of the river has virtually eliminated sand supply to the lower or coastal plain reach of the river. Urban development of areas adjacent to the lower reaches of the river and also the composition of the flat alluvial plain further reduces the potential of much sand supply to the shore. Present shoaling at the San Gabriel ocean outlet works is indicated to be from littoral processes, and there is little evidence of material

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[2] Data from "Beach erosion control report on cooperative study of Orange County, California," Appendix V, Phase 2. U.S. Army Engineer District, Los Angeles, Calif., March 1962; House Doc. 602, 87 Cong., 2d Sess.

[3] HERRON, W. J., JR., and HARRIS, R. L. NEW METHODS OF CONSERVING BEACH SAND. Shore and Beach 30 (1): 34-37. 1962.

[4] MCOUAT, H. W. HISTORY OF LOS ANGELES HARBOR. Coastal Engin. Conf. Proc. 1, Council on Wave Res. Univ. of Calif. 1950.

[5] KENYON, EDGAR C., JR. HISTORY OF OCEAN OUTLETS, LOS ANGELES COUNTY FLOOD CONTROL DISTRICT. Coastal Engin. Conf. Proc. 1, Council on Wave Res., Univ. of Calif. 1950.

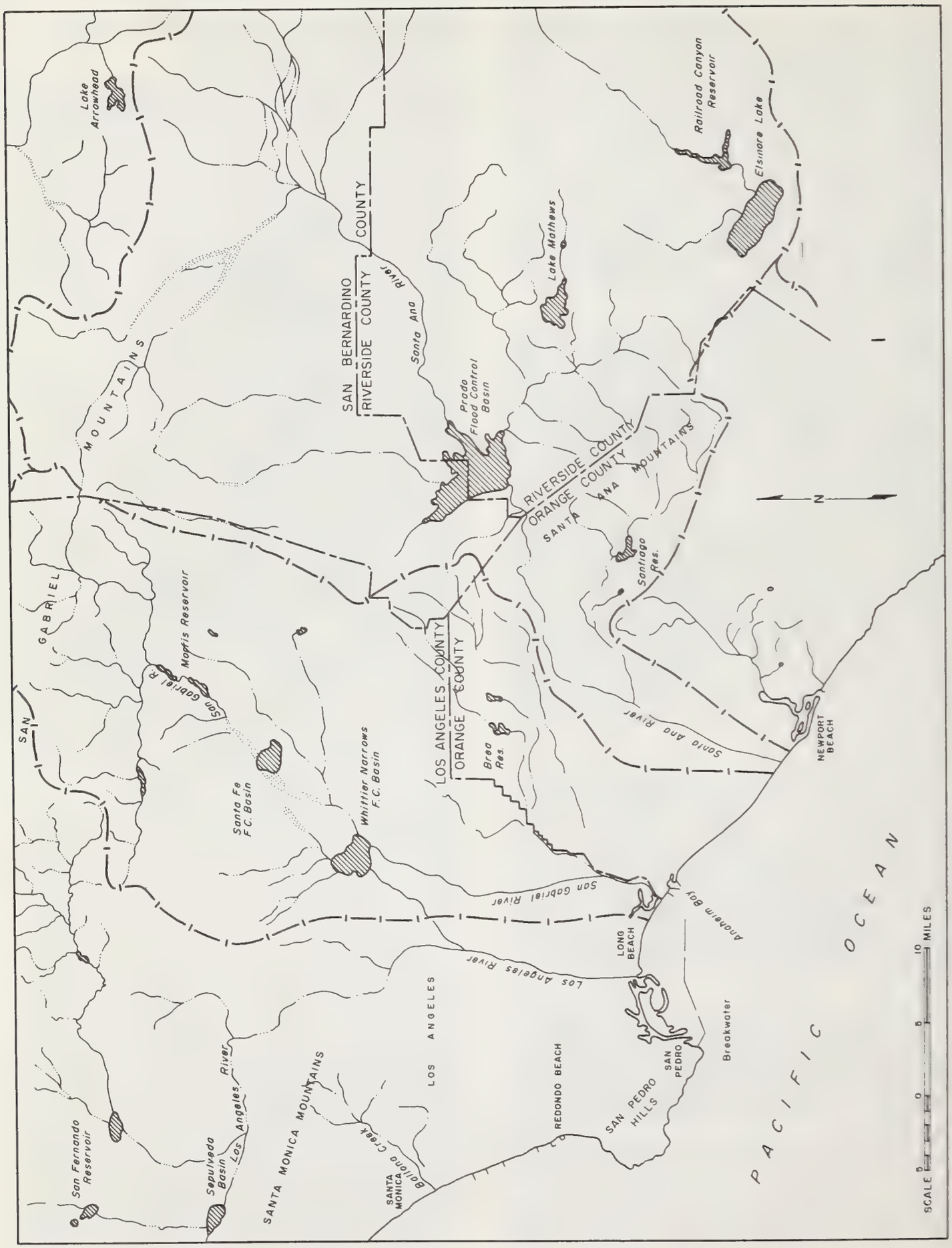

FIGURE 4.—Drainage areas of Los Angeles, San Gabriel, and Santa Ana Rivers, Calif.

supply from the river discharge. It could be reasoned if the San Gabriel River did supply material to the coast, this supply has now been virtually eliminated by the construction and development of works on the upland reaches of the river.

The Anaheim Bay entrance jetties are located about 1 mile east, or downcoast, of the San Gabriel River outlet. Sand fill was placed along the shore between the San Gabriel River and North jetty at Anaheim in 1959, and this shore sector has been fairly well stabilized. The Anaheim Bay entrance jetties were completed in 1944 and, due to their length, effectively interrupt any littoral material movement from the west. The shore immediately east of these jetties receives essentially no littoral supply from the west and as a result very serious erosion occurs. The erosion problem is progressing from the Anaheim Bay east jetty to Newport Beach, a shoreline distance of about 13 miles.

The Santa Ana River mouth is about 11 miles southeast of Anaheim Bay and within the problem area. The drainage basin is shown on figure 4. The basin area is about 2,470 square miles. The river headwaters are in the San Bernardino Mountains. There are no known data on the amount of sediments transported to the coast, but at the present time it is apparently negligible. Upland development has restricted the flow of the river in its lower reaches to the point that very little beach material reaches the shore. Except during the rainy season (November to March) the discharge is insufficient to breach the barrier formed by littoral processes at the mouth of the river.

Discussion

Although certain structures built along the shore between San Pedro and Newport have had varying effects on the shore, the main cause of the erosion problems is the reduction of sand supply to the shore from upland river flow. There is very little littoral drift transported around Point Fermin to the Long Beach area as far as it is known. The littoral drift moving from Anaheim Bay to Newport Beach is eventually deposited in the Newport Submarine Canyon located at the entrance to Newport Harbor. Thus, the general picture is one where no alongshore supply enters the system from the west, and the supply of material now existing within the system will be eventually lost at the eastern end. Since essentially all sand supply from upland river flow has been eliminated, shore erosion will continue unless the deficiency of alongshore material supply is made up by some other means.

One of the very important factors in considering various plans for stabilizing the shore between Anaheim Bay and Newport Beach was the limited availability, within reasonable distances, of an inland source of supply of suitable sand for initial beach restoration and periodic nourishment. In this respect, seawall and revetment along the 12½-mile reach of shore was considered but estimates of cost for either of these structures indicated them to be substantially higher as compared to other plans. Stabilizing the shoreline by a seawall or revetment would also not provide adequate widths of beach for recreational purposes. Evaluation was made for a plan involving beach fill and periodic nourishment, with and without a system of groins, and it was found that the initial fill and periodic nourishment requirements exceeded the availability of sand located in all nearby borrow areas. Another plan was considered that involved beach restoration and a series of offshore breakwaters parallel to the shore. Such a system would, in effect, compartment the shore. Each breakwater would impound the southerly moving littoral drift in its lee, and periodically the impounded material would be mechanically transferred back upcoast to a point just below the influence of the next upcoast breakwater. Although this plan would provide a long range solution to the problem, a more economical plan was developed and recommended that involved only one offshore breakwater located at the upcoast limits of Newport Beach and a feeder-type beachfill placed just below Anaheim Bay Harbor. This plan of improvement is shown on figure 5.

The fill, after being transported downcoast and impounded by the breakwater, would periodically be mechanically transferred back upcoast and deposited in the feeder beach zone just below Anaheim Bay Harbor.

Basically, this plan involves the periodic rehandling of the same sand supply. It was estimated that nearby borrow areas contained sufficient quantities of suitable materials to construct the initial feeder beach and to replace normal losses to the offshore zone that would occur between the feeder beach and the breakwater. The initial beach fill below Anaheim Bay Harbor would be about 3 million cubic yards. The downcoast littoral drift rate is about 300,000 cubic yards per year; thus, erosion of the fill zone would be about equal to this rate as well as the impoundment rate behind the offshore breakwater at Newport Beach. The offshore breakwater will provide a sheltered area from which a conventional pipeline dredge may operate. If 5-year increments are considered, this floating plant, along with a series of booster pumps located along the shore, would need to transfer 1,500,000 cubic yards in each operation. The sand-transfer operation could also be accomplished with other types of floating plant

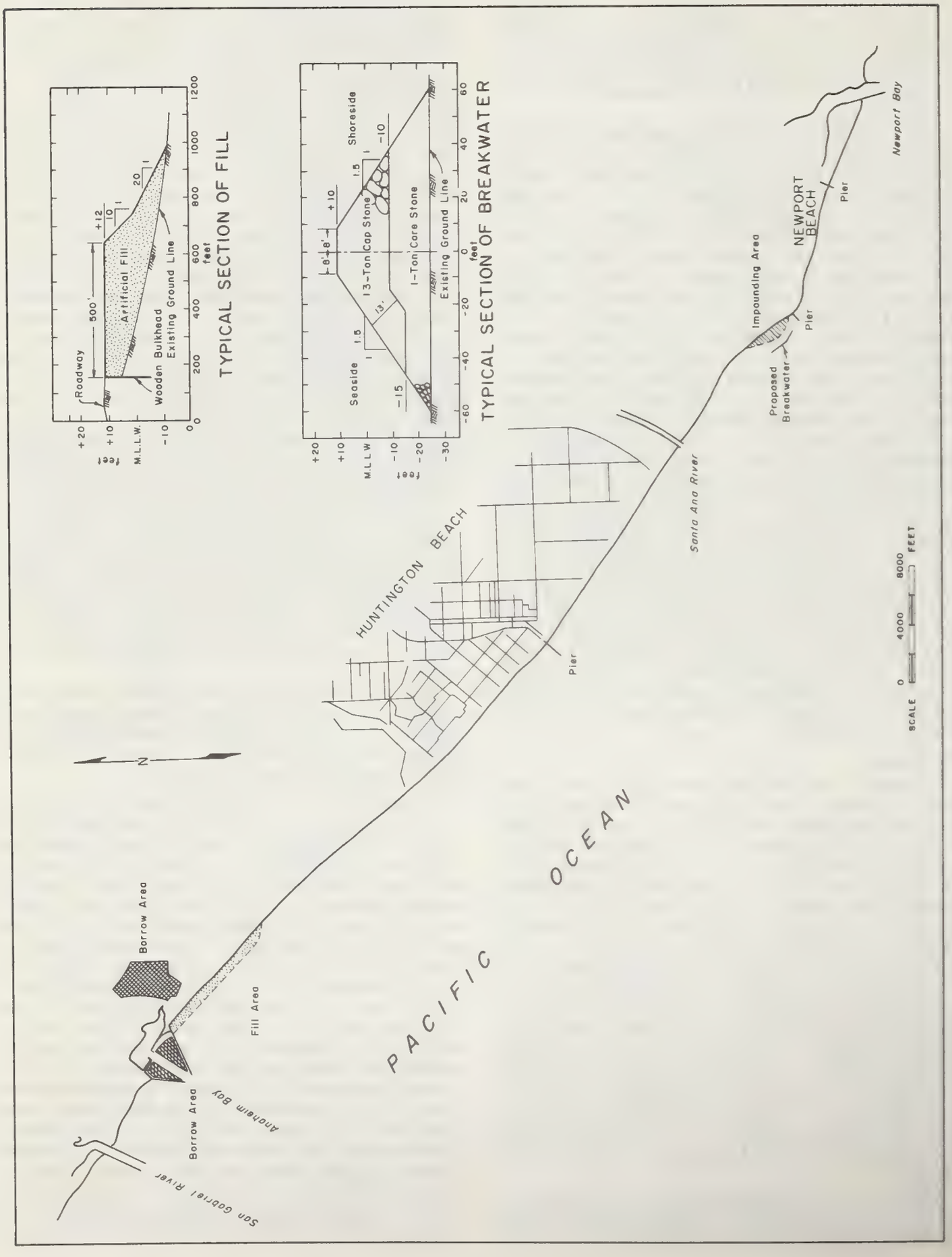

FIGURE 5.—General plan of improvement, Anaheim Bay to Newport Beach, Calif.

such as a hopper dredge with pumpout capability. It was estimated 50,000 cubic yards per year would be lost to the offshore zone out of the system between the feeder beach and the breakwater. Therefore, on a basis of 5-year intervals between pumping operations, 250,000 cubic yards would need to be obtained from other borrow sources and supplemented to the feeder beach zone to maintain complete shore stability for the problem area.

Construction of this plan of improvement has not been initiated as of this date (January 1963). The plan is unique and, when implemented, its functional operation will undoubtedly be carefully viewed by many engineers, as the project will provide valuable guidance on this approach to shore stabilization.

Concluding Remarks

The improvement and construction of works in drainage basins tributary to a coast, by their very nature, tend to reduce the quantity of sediments reaching the coast. This is particularly true of the sand-size class of sediments which serve to nourish the beach zone. The objective of full basin development is to achieve optimum economic use of all upland resources. The advantages from predicting effects to the coastal zone during the planning stage of basin development are apparent. If detrimental effects will prevail, early corrective measures can be implemented at the coastal zone. Such early action will prevent the shore problem from reaching a critical condition and, in general, will have substantial effect on the economy of the area.

If specific sedimentation data are not available relative to the upland basin development, the engineer must deal with the factors pertinent to the coastal problem and develop his solutions accordingly. He must explore and study all information pertinent to sediment contributed to the coast by upland rivers and streams prior to, during, and after construction of stabilizing works on the upland drainage basin. Detailed criteria and up-to-date techniques for the planning and design of coastal works are presently available;[6] however, such guidance leading to ultimate structural design of coastal works has little value if realistic evaluation is not made of the past, present, and future supply of materials to the littoral zone from not only upland supply but from all sources. The economic justification for constructing certain stabilizing works along a shore are frequently based on a time increment of 50 years and in some cases on a 100-year basis. Therefore, present and projected supply of sediments from upland sources may, and generally will, be a significant factor in the functional and structural design within the plan of improvement for the coastal problem area in question.

[6] U.S. ARMY CORPS OF ENGINEERS, BEACH EROSION BOARD. SHORE PROTECTION PLANNING AND DESIGN. Tech. Rpt. 4, 243 pp. Rev. ed. 1961.

SAND MOVEMENT ON COASTAL DUNES

[Paper No. 75]

By J. W. JOHNSON, *professor of hydraulic engineering, University of California, Berkeley*

Introduction

The problems of the supply and loss of sediment at a shoreline are of considerable importance at many localities along the coastlines of the United States. One basic mechanism involved in this overall problem is the transportation of sand by wind action (*5*). For example, at many localities along the Washington, Oregon, and California coasts considerable amounts of sand apparently are moved inland each year by wind action (*3, 8*). A study at the major dune areas of the California coast (*8*) shows that three basic conditions are satisfied for the existence of a dune system: (1) a large supply of sand from a nearby major stream; (2) a shoreline orientation approximately parallel to the crests of the prevailing wave condition, thus creating a favorable condition for a low littoral current and consequently a location for sediment accumulation; and (3) low topography back from the beach where the prevailing onshore winds can easily move the sand inland from the region of accumulation at the beach. Figure 1 is a typical example of a major dune area on the California coast.

To obtain reliable data on the approximate magnitude of the annual supply or loss of sand from a coastline by wind action, a program of laboratory studies was undertaken in a special wind tunnel with sands of various sizes to define the relation between the rate of sand transport and the character of the wind and the sand. This laboratory program was supplemented by a limited program of field studies that was made to define the extent of the dune areas, the character of the dune sand, and the frequency of occurrence of coastal winds capable of moving sand. Thus, to provide information on the character of the sands encountered in actual dune systems and therefore to serve as a basis for the sand sizes that should be

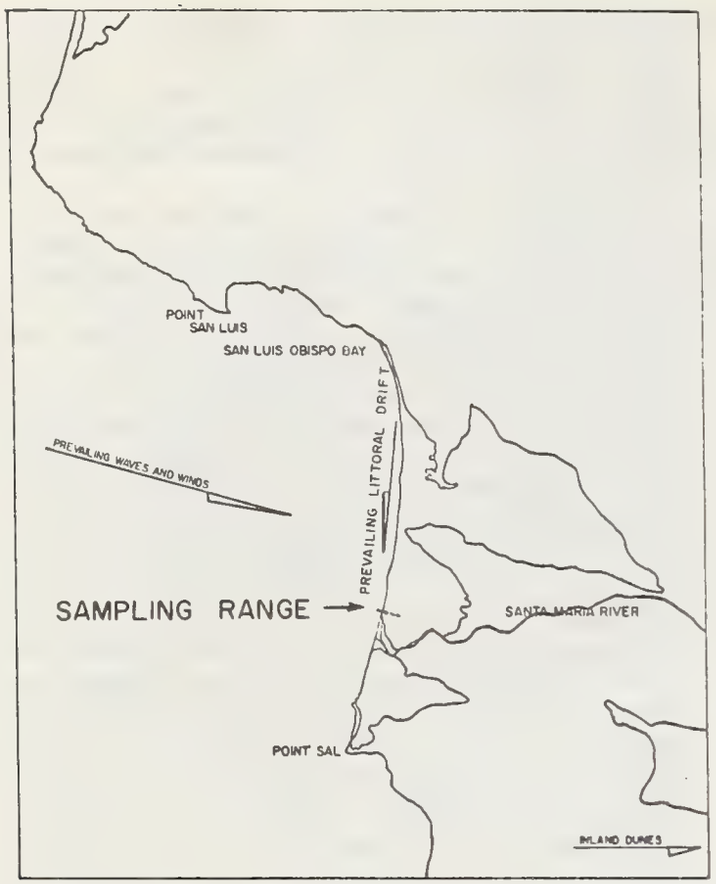

FIGURE 1. — Example of usual shoreline configuration for major dunes on the California coast.

studied in the laboratory, a sampling program was conducted at most of the important dune areas on the California coast. Surface samples were taken at the midtide level on the beach face and on a range extending inland in the direction of the prevailing onshore winds. Typical of such information is that shown in figure 2,

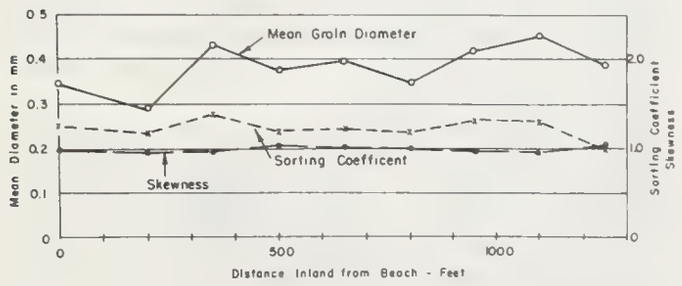

FIGURE 2. — Variation of mechanical composition of dune sand with distance from beach (near Santa Maria River mouth).

which was obtained along the range indicated in figure 1. This plot shows the mean grain size, sorting coefficient, and skewness as a function of distance from the shoreline.

Considerable work has been done on the mechanics of sand movement by wind by numerous investigators (*1, 6, 9*). Emphasis in these past studies was on the distribution of windspeed above the sand surface (with and without sand movement) and the rate of sand movement as a function of the principal variables.

The shear stress, T, produced at the sand surface by airflow, is one of the most important factors in the basic mechanics of sand movement by wind. When the shear stress exceeds a critical value, the sand particles start to move. The windspeed profile and shear velocity, U_*, are the primary factors involved in sustaining sand movement — the shear velocity being defined as $\sqrt{T/\rho}$ where ρ is the density of the air. Bagnold (*1*), in his studies of the initiation of sand movement, derived the following expression for the threshold value of the shear velocity,

$$U_{*t} = A\sqrt{\frac{\sigma - \rho}{\rho} gd} \qquad (1)$$

where d is the grain diameter, g is the acceleration of gravity, and σ and ρ are the densities of sand and air, respectively. Bagnold found that the coefficient A had a nearly constant value of 0.1 for a sand diameter of 0.25 mm. and greater.

Several investigators have developed expressions for the rate of sand movement as a function of certain variables. A few of these expressions are as follows:

Bagnold formula (*1*). — The rate of sand movement per unit width and unit time, q, is given by

$$q = C\sqrt{\frac{d}{D}}\frac{\rho}{g}U_*^3 \qquad (2)$$

where D is the grain diameter of a standard 0.25-mm. sand, d is the grain diameter of the sand in question, ρ is the density of the air (in c.g.s. units, $\frac{\rho}{g}$ is equal to 1.25×10^{-6}), U_* is the shear velocity, and C has the following values:

1.5 for a nearly uniform sand;
1.8 for a naturally graded sand;
2.8 for a sand with a very wide range of grain sizes.

Kawamura formula (*6*). — The rate of sand movement, q, is given by,

$$q = k\frac{\rho}{g}(U_* - U_{*t})(U_* + U_{*t})^2 \qquad (3)$$

where ρ is the density of air, U_* is the shear velocity, U_{*t} is the threshold shear velocity, and k is a constant that must be determined by experiment. For a sand of average grain size, 0.25 mm., Kawamura obtained k=2.78 in wind tunnel tests, with all terms being in c.g.s. units.

Experimental results obtained in wind tunnel tests by Bagnold (*1*) and Kawamura (*6*), using

a sand grain diameter of 0.25 mm., present widely differing results as shown in figure 3.

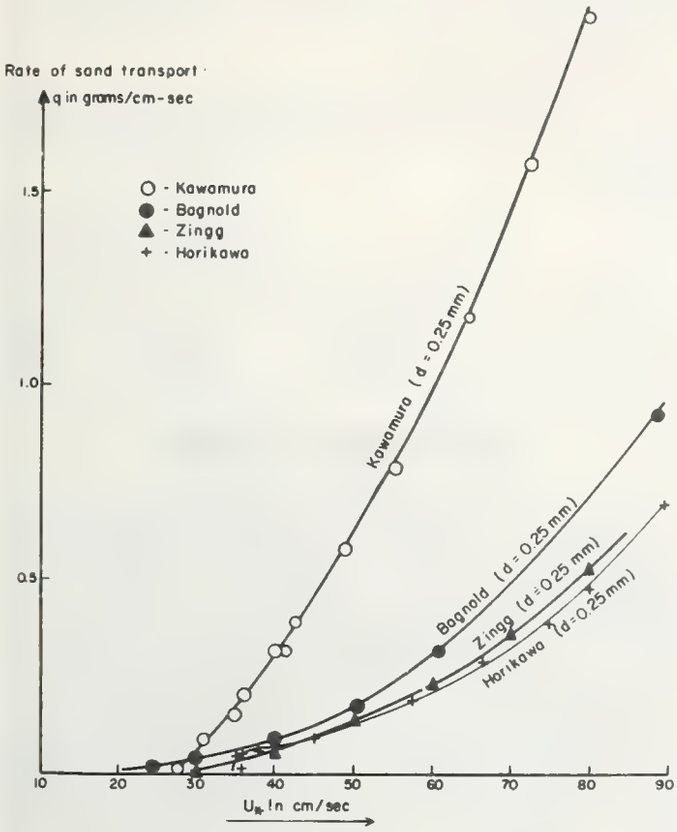

FIGURE 3. — Experimental data on sand transport determined by previous investigators.

Also plotted in this figure are the results of wind tunnel tests by Zingg (*9*) and Horikawa and Shen (*4*) for a grain diameter of 0.25 mm. From his results, Zingg (*9*) modified the Bagnold formula to

$$q = C\left(\frac{d}{D}\right)^{3/4}\frac{\rho}{g}U_*^3$$

where C has the value 0.83.

In addition to the above formulas, O'Brien and Rindlaub (*7*) proposed the following formula from data derived by field tests:

$$G = 0.036\,U_5^3 \text{ (for } U_5 > 20 \text{ ft./sec.)}$$

where G is the rate of transport in pounds per day per foot width, and U_5 is the wind velocity 5 ft. above the sand surface in feet per second.

Confirmation of the above formulas by field results is not particularly good, but since there is considerable scatter in the experimental data, these formulas are still useful in the description of a particular condition when a suitable constant is selected.

Experimental Program

In order to reconcile some of the apparent differences in the various existing relationships for the rate of sand movement and to consider those conditions usually found in the field, a program of laboratory studies was made at the University of California under sponsorship of the Beach Erosion Board. The first tests in this program were made to develop a suitable sand trap for measuring the rate of sand transport in the field. These studies have been reported elsewhere (*4*) and are not summarized in this paper. Subsequent to these earlier tests, which were conducted in a wind tunnel 12 in. by 15 in. in cross section, a larger and more suitable tunnel 4 ft. wide by 2.5 ft. high and 101.5 ft. long (fig. 4) was constructed and used in the laboratory tests discussed below (*2*).

Wind in the tunnel was generated by a variable speed fan at the exit end. Windspeeds up to a maximum of about 40 ft./sec. could be attained. Vertical and horizontal velocity distributions during tests were made with a standard Prandtl-type pitot tube connected to a previously calibrated Magnehelic gage having a range of one-half inch of water and graduated into 0.02-inch divisions. During all tests, sand was spread over a length of about 62 ft. of the

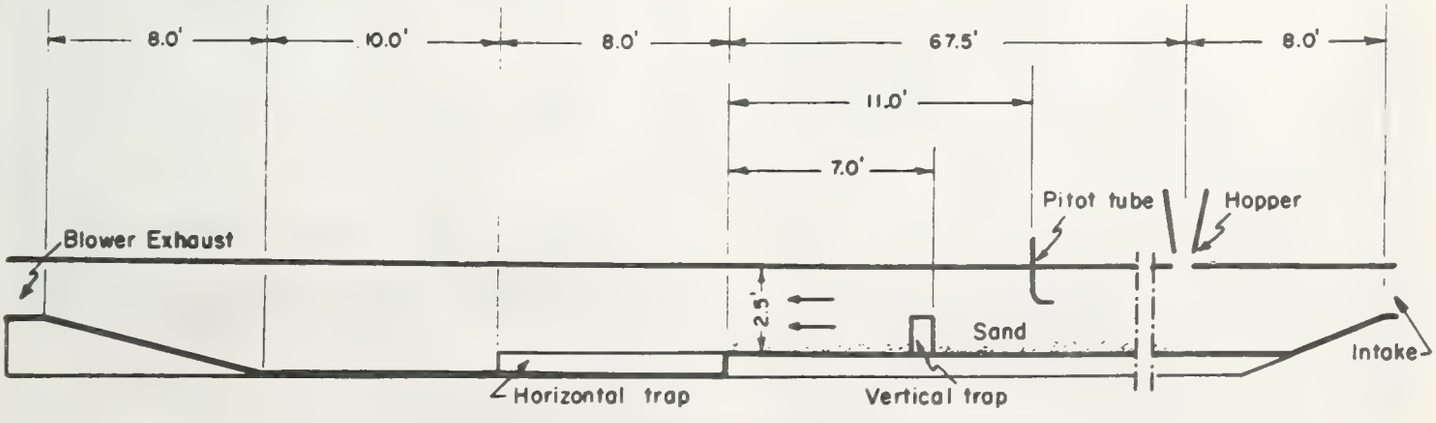

FIGURE 4. — Wind tunnel used in sand transport studies.

flume with a depth of 2 inches. A hopper to feed sand into the flume at a uniform rate was placed near the entrance to the tunnel (fig. 4). The rate of sand feed was adjusted to be equal to the rate of sand transport as measured by the sand traps, which consisted of the vertical trap developed by Horikawa and Shen (*4*) and a horizontal trap. This latter trap consisted of 18 compartments permanently fixed at the end of the sand bed (fig. 4).

In order to eliminate side-wall effects with the horizontal trap, the rate of sand transport was measured only over a width of 2 ft. in the central part of the flume. Sand was removed from the compartments in the horizontal trap at the conclusion of each run with a vacuum cleaner. Comparison of the rate of transport as measured by the vertical trap with that measured with the horizontal trap showed that the vertical trap is close to 100 percent efficient at windspeeds higher than about 27 ft./sec., but it becomes much less efficient at lower speeds (*2*). For high windspeeds, the grain-size distribution of the sand caught in the vertical trap was very close to the grain-size distribution of the bed except for the very largest grains (*2*). The relative absence of these larger grains probably is caused by the platform of the trap, which sometimes can be an obstacle to surface creep.

For windspeeds near the threshold value, the grain-size distribution of sand in the vertical trap shows a distinct lack of the larger grains. This condition cannot be attributed entirely to the inefficiency of this trap but probably indicates the manner in which sand is moving near the threshold; i.e., the larger grains are not in general movement.

The mechanical composition of the three sands used in the tests is shown in figure 5. The principal characteristics of these sands are as follows:

Sand	Mean grain diameter (mm.)	Sorting coefficient	Skewness
A	0.44	1.23	1.01
B	.30	1.15	1.00
C	.145	1.24	.98

Experimental Results

Rate of Sand Transport

The amount of sand caught by the horizontal trap was measured for velocities varying from the threshold value to about 37 ft./sec. The feeding of sand into the wind tunnel at the upstream end of the sand bed was an important factor in sand movement for the lower wind velocities. Feeding sand into the tunnel during tests appreciably lowers the threshold velocity and, at the same time, changes the amount of sand transported for lower velocities. This effect is evident in figure 6, which shows the rate of transport as a function of wind velocity (and shear velocity) with and without sand

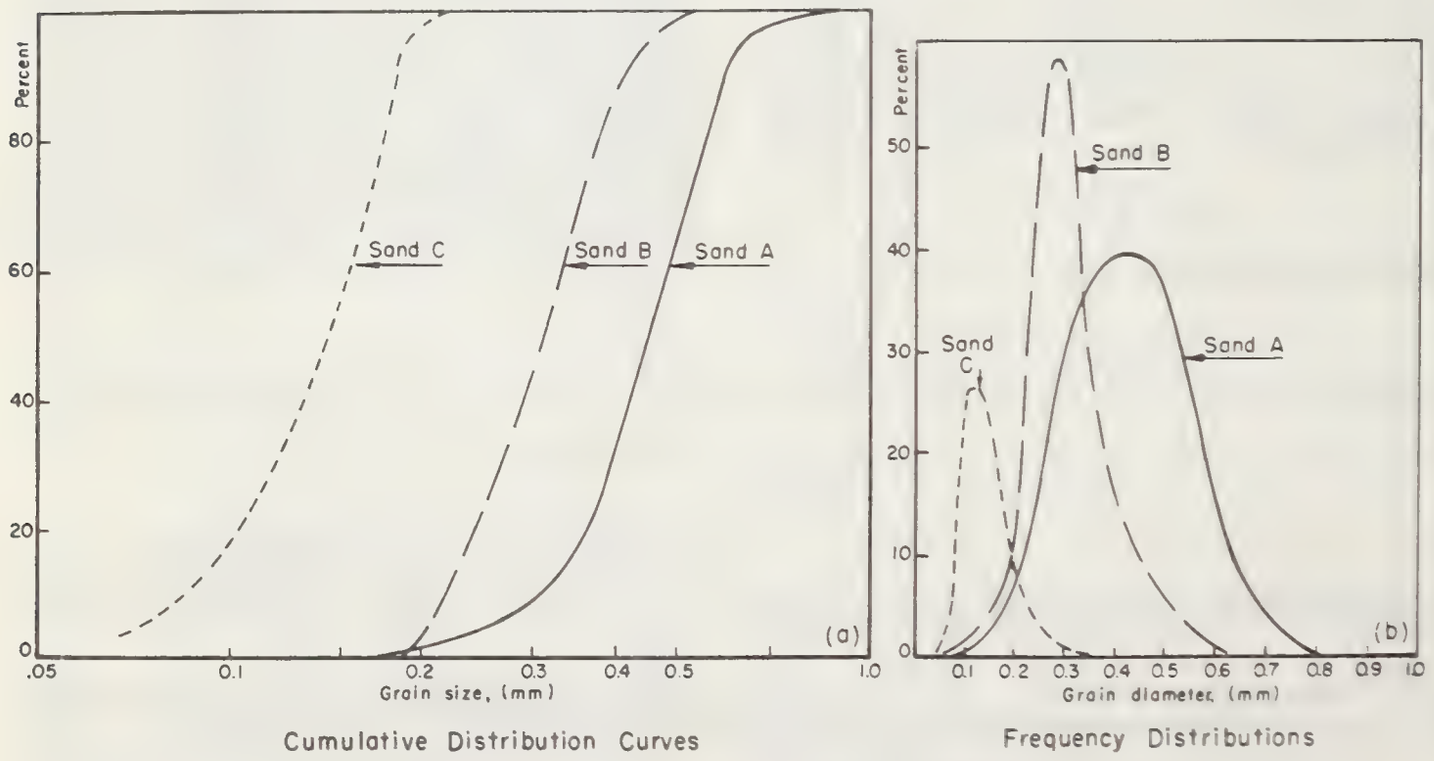

FIGURE 5. — Mechanical composition of sands tested.

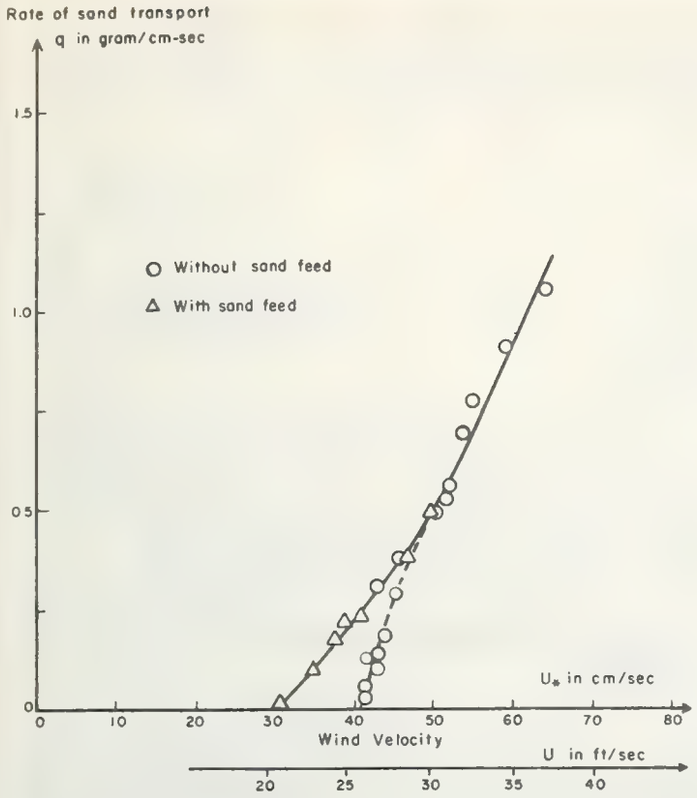

FIGURE 6. — Experimental data on the rate of transport of sand A (d = 0.44 mm.) with and without a sand feed.

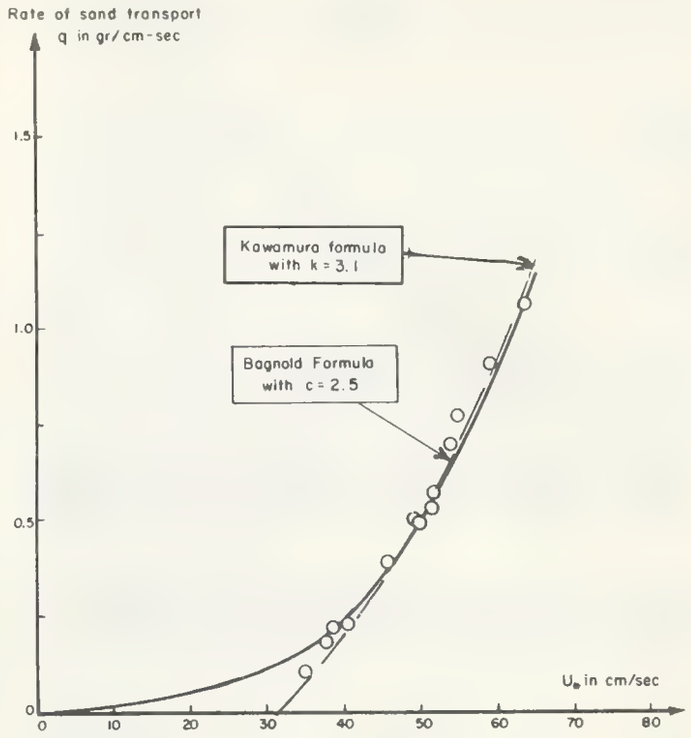

FIGURE 7. — Comparison of experimental data with the Bagnold and Kawamura formulas for sand A (d = 0.44 mm.).

feed for sand A (d=0.44 mm.). The apparent reversal of the curve obtained without a sand feed is perhaps due to the fact that the sand used in this study had a relatively wide range of grain sizes (fig. 5). At or near the threshold condition, it is possible that the action of the smaller grains was impeded by the larger grains, thus modifying the overall values for the threshold velocity and the rate of transport. More precisely, near the threshold the sand grains move mainly by saltation. Since the surface layer remains practically immobile (no surface creep), the smaller grains are hidden by the larger ones and as a result the sand behaves as though it had a much larger mean diameter.

The experimental values for the maximum rate of transport (i.e., with a sand feed), q, are compared to the values predicted from the Bagnold (*1*) and Kawamura (*6*) formulas as follows.

Sand A (d=*0.44 mm.*) — The experimental value of the threshold shear velocity (with sand feed) was 30 cm./sec., which compares favorably with a value of 34 cm./sec. as calculated by the Bagnold formula (equation 1). For the rate of transport, Bagnold (equation 2) proposes a value of C=1.8 for normally graded sand and C=2.8 for a sand with a very wide range of grain size. If a value of C=2.5 is used for sand A, the experimental and calculated values are as shown in figure 7. From this figure, except for wind velocities near the threshold, where the Bagnold formula is not applicable, the agreement is good. In the Kawamura formula (equation 3) with a value of U_{*t}= 30 cm./sec. combined with a value of k=3.1, excellent agreement between experimental and calculated values was obtained over the entire range of shear velocity, as shown in figure 7.

Sand B (d=*0.33 mm.*). — The experimental value of the threshold shear velocity (with sand feed) was 16 cm./sec., which agrees fairly well with a value of 18 cm./sec. as calculated with Bagnold formula. With respect to rate of transport, figure 8 was prepared to show a comparison of experimental values with the Bagnold and Kawamura data on a sand of 0.25 mm. in diameter. It is evident that the curves differ considerably, although the sand in each case had almost the same mean grain diameter. The sorting coefficient of the various sands possibly was quite different. In the study with sand A (d=0.44 mm.) the Bagnold and Kawamura formulas could be used to describe the experimental data; however, with sand B no values for the constants C and k could be found to describe the rate of transport over the entire range of shear

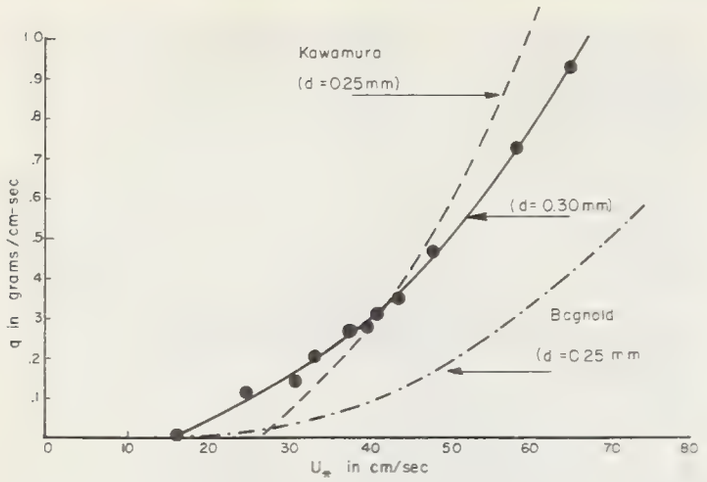

FIGURE 8. — Comparison of experimental data on sand B (d = 0.30 mm.) with the experiments of Bagnold and Kawamura on a sand 0.25 mm. in diameter.

velocities. Those values that most closely described the experimental data are shown in figure 9.

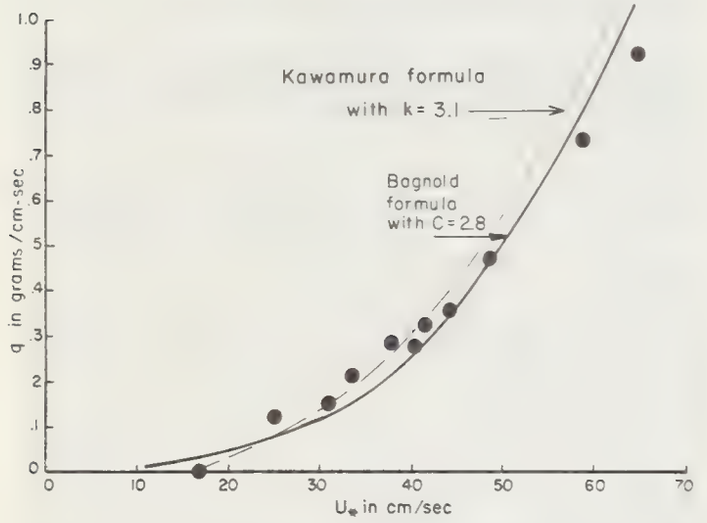

FIGURE 9. — Comparison of experimental data with Bagnold and Kawamura formulas for sand B (d = 0.30 mm.).

Sand C (d=*0.145 mm.*). — The experimental value of the threshold shear velocity (with sand feed) was 22.0 cm./sec. and that calculated by the Bagnold formula was 17.8 cm./sec. This relatively large difference between the observed and calculated values of the threshold velocity is expected from the work of Bagnold (*1*), wherein it was shown that for such small grain sizes as sand C the coefficient A is not a constant and the shear velocity reaches a minimum. For the rate of transport, figure 10 shows the experimental values compared with the Bagnold and Kawamura formulas. In this comparison, a value of C=1.5 was used in the Bagnold formula. This is the lower limit of the value of C recommended by Bagnold and applies to a nearly uniform grain size. For the Kawamura formula, a value of k=1.0 had to be selected in order that the calculated curve would pass near the experimental points.

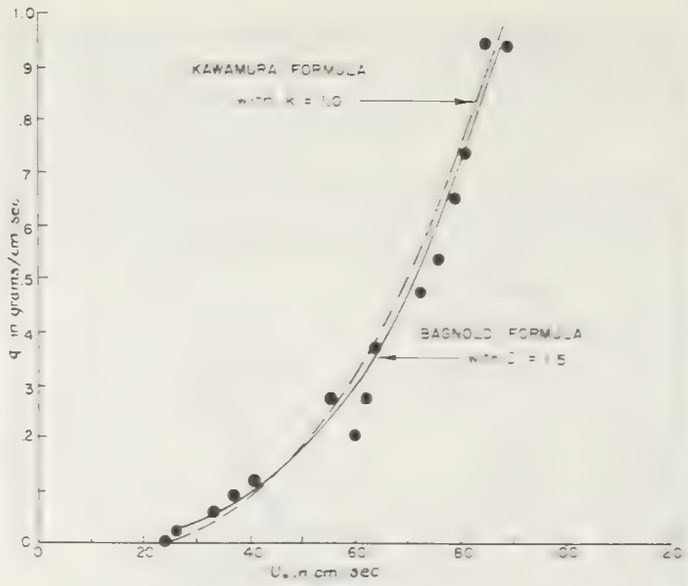

FIGURE 10. — Comparison of experimental data with Bagnold and Kawamura formulas for sand C (d = 0.145 mm.).

Response Time of Sand Bed to Change in Windspeed

In order to investigate the response time of the bed to a change of windspeed, some tests were made in which the wind was first allowed to blow over the sand surface (using sand A) at a velocity of 31.5 ft./sec. for a relatively long time (sufficiently long to observe a duplication of the results on the amount of sand transported, both in the vertical and horizontal traps). The windspeed then was suddenly changed to a value of 35 ft./sec. and the sand collected in the vertical trap was weighed every 2 minutes, until a new constant rate of transport was observed. The windspeed then was again adjusted to the first value, while the same measurements were made with the vertical sand trap. After a sufficiently long time, the windspeed was again adjusted to the higher value, and the same process repeated.

There was some scatter in the data, especially for the higher velocity. This dispersion probably was due to some extent to the inaccuracy of the windspeed readings, these speeds being slightly different in corresponding runs. But as the important fact is the rate of transport with respect to time, the part of the dispersion due to differences in the mean windspeed can be eliminated by considering the discrepancy between the measurements made within the first

few minutes of each run and the average of the last measurements, when the equilibrium is reached. Figure 11 shows the rate of sand transport as a function of time with this correction.

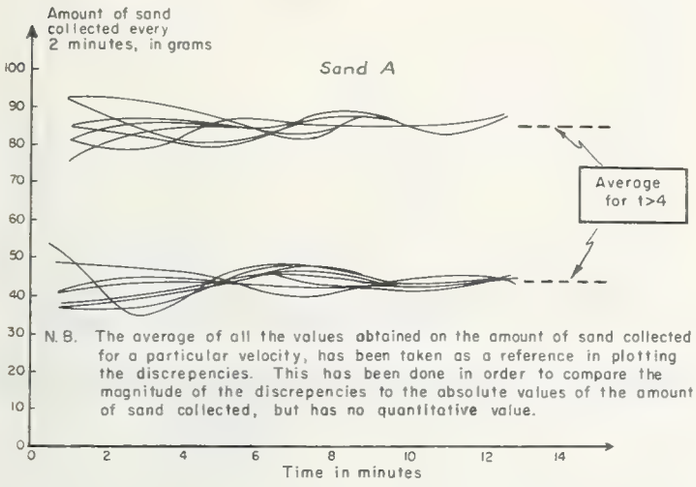
Amount of sand collected every 2 minutes, in grams
Sand A
Average for t>4
N.B. The average of all the values obtained on the amount of sand collected for a particular velocity, has been taken as a reference in plotting the discrepencies. This has been done in order to compare the magnitude of the discrepencies to the absolute values of the amount of sand collected, but has no quantitative value.
Time in minutes

FIGURE 11. — Change in rate of transport with time following a change in wind velocity.

The dispersion was greater at the beginning than at the end of a run, but except for a slight increase during the first 4 minutes in most of the runs, no trend could be observed. In general it appears that the rate of transport adjusts itself rather quickly to a new windspeed. Consequently, under field conditions a knowledge of the actual duration of windspeeds of various magnitudes apparently should permit the calculation of the probable volume of sand moved on a given dune system in a given period of time.

Effect of Moisture on Sand Transport

Generally moist sand found along coasts can be classified into two categories, depending upon the origin of the moisture:

(1) *Moist sand that has collected moisture from the atmosphere.* — Unlike very fine particles such as dust or loess, the sand does not appear to absorb moisture readily. As indicated below, however, there exists a correlation between the water content of the sand and air humidity. Since there is such a correlation and since in the field it is easier to measure the air humidity than the sand water content, the air humidity instead of the water content of the sand was taken as the variable in the studies discussed below.

(2) *Moist sand whose water comes from sources other than air humidity.* — Such sources are rain, rising of underground water, and sea water remaining in the sand by wave or tide action. When a wind not saturated with water vapor blows over such a sand, it gradually dries out the surface layer of the sand bed until an equilibrium is reached between humidity of the wind and the water content of the surface layer of the ground. The study of sand movement is relatively complex in this case, where water contents of air and sand, as well as wind duration, are factors.

To obtain information on some of the above problems, Belly (*2*) performed a series of experiments on sand A (d=0.44 mm.) in a closed-circuit wind tunnel where humidity could be controlled. The first series of tests were concerned with the determination of the effect on the threshold velocity where the moisture content of the sand was relatively low. In these tests, U_* was determined from a series of vertical velocity distribution curves for different mean velocities and different air humidities, h. Figure 12 shows the relation between the shear

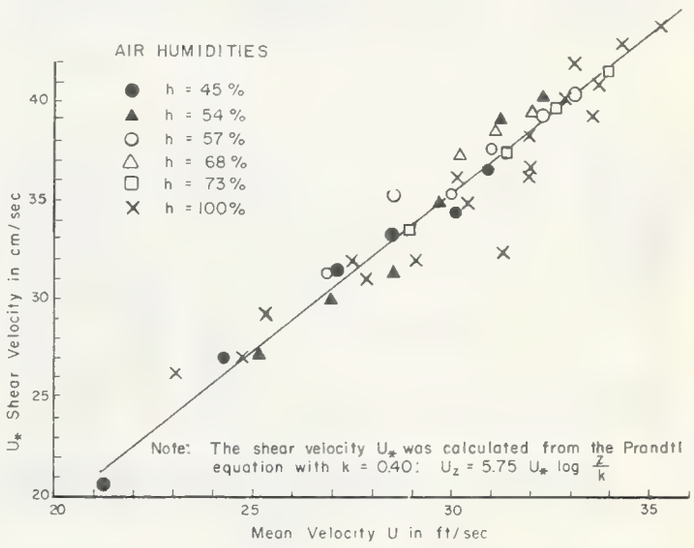
AIR HUMIDITIES
h = 45 %
h = 54 %
h = 57 %
h = 68 %
h = 73 %
h = 100%
U* Shear Velocity in cm/sec
Note: The shear velocity U* was calculated from the Prandtl equation with k = 0.40: Uz = 5.75 U* log z/k
Mean Velocity U in ft/sec

FIGURE 12. — Relation between mean velocity and shear velocity for various air humidities.

velocity, U_*, and the mean velocity, U. This relation is quite consistent and can be expressed by a straight line. Thus, it can be stated that the wind drag is practically uninfluenced by air humidity. In the later tests, only the value of the mean velocity was recorded and the value of U_* was calculated from this graph.

The water content of sand obtained during the various runs is plotted against the corresponding air humidity in figure 13. The scatter of data is important, but there is a general tendency for the points to follow along a straight line (which obviously should pass through the origin).

The threshold velocity was found by investigating the lower part of the curves for the rate of sand transport. The three sample curves in figure 14 clearly indicate the change in the rate of sand transport with humidity. From such curves, the data on the variation of the threshold shear velocity distinctly show an increase

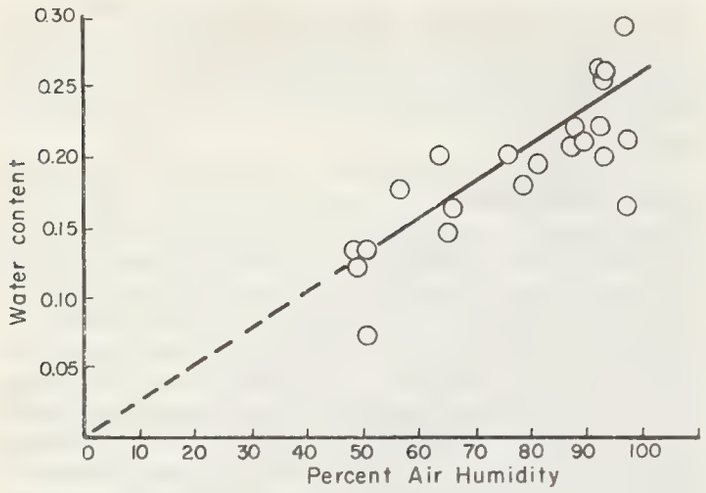

FIGURE 13. — Relation between water content of sand and air humidity.

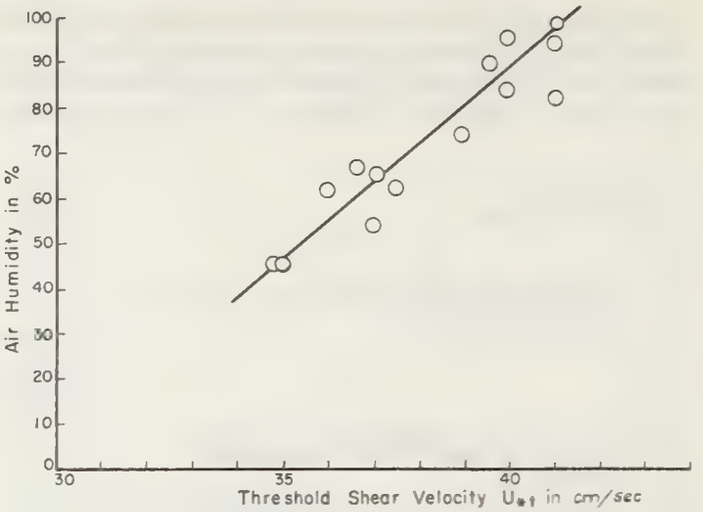

FIGURE 15. — Relation between threshold shear velocity and air humidity (sand A).

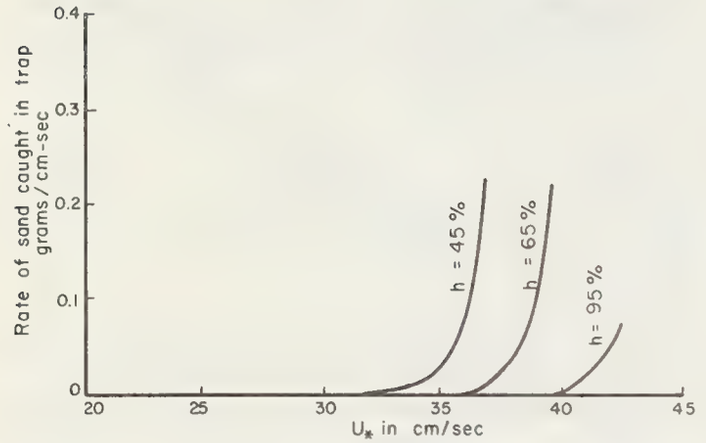

FIGURE 14. — Variation of rate of sand transport with air humidity.

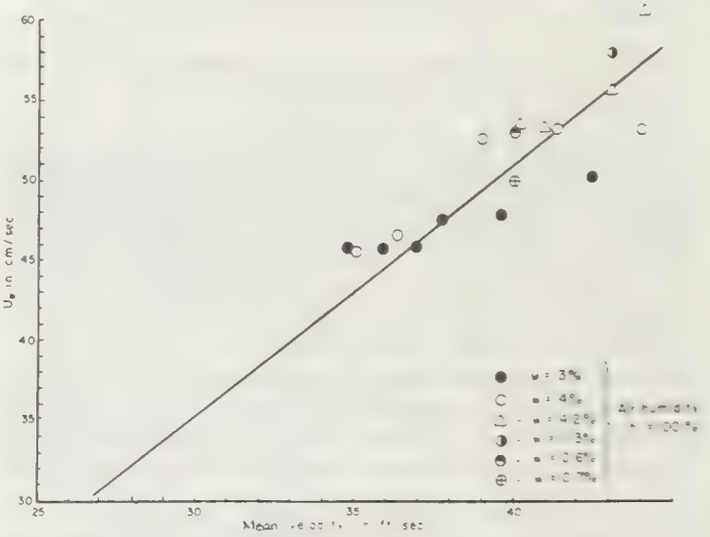

FIGURE 16.—Relation between shear velocity and mean velocity for high water contents.

with air humidity, and the relationship is nearly linear (fig. 15).

The second series of tests by Belly (*2*) on threshold velocity were concerned with sand of high water content; that is, greater than 0.25 percent. The relation between U and U_* under such conditions is shown in figure 16 and appears to be linear and merely an extension of the data shown in figure 12. During the tests in which these data were obtained, the air was maintained saturated, and the water content, w, of the sand was varied.

The tests showed that with a relatively high water content of the sand (in excess of 1 percent), the windspeed necessary to initiate sand movement becomes increasingly important. One explanation for this increase is that the sand surface becomes very smooth under wind action. The water contained in the sand fills the interstices between the grains and makes the extraction of the grains by the wind much more difficult than when dry. The experiment could not be made for a water content higher than 4 percent because the wind strength necessary to initiate the movement could not be obtained with the available equipment; however, it might be expected that the wind strength would increase very rapidly with an increase of water content. It is even probable that for a very high water content (flooded sand) the problem changes aspect and becomes closer to the problem of an interface between two fluids.

Using the relationship between water content and air humidity in figure 13, it is possible to complete the results of the study of low water content and thus find a relationship (fig. 17) between U_{*t} and w for the total range of water content (0 to 4 percent). As to be expected, since the air humidity in itself does not play an important role in the sand movement, there is no break at the point that joins the two sets of

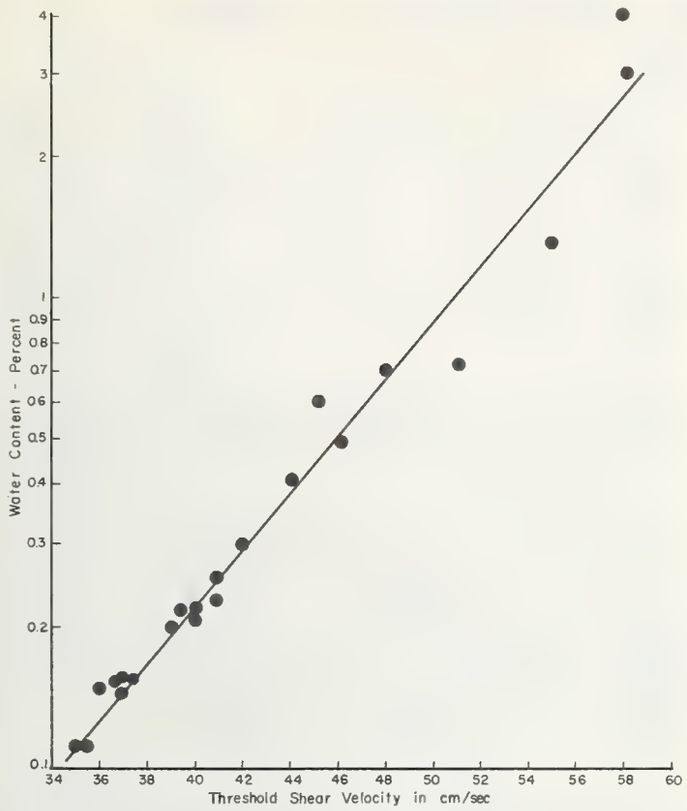

FIGURE 17. — Variation of the threshold shear velocity with moisture content (from figs. 12 and 16).

data; that is, at a threshold shear velocity of 40 cm./sec. The data follow a straight line, thus indicating that the relation between U_{*t} and w can be put into the form,

$$U_{*t} = a \log_{10} w + b$$

where a and b are two constants obtained from the graph. It was found that

$$U_{*t}=17\ (\log_{10} w+5.1)\ \text{cm./sec.}$$

or $$U_{*t}=28\ (0.6 \log_{10} w+1.8)\ \text{cm./sec.}$$

where 28 is the value of U_{*t} in centimeters per second given by the Bagnold formula.

If it is assumed that moisture affects the movement of sand of different grain sizes in the same manner, the Bagnold formula for the threshold shear velocity may be modified as follows:

$$U_{*t}=A\sqrt{\frac{\sigma-\rho}{\rho} g\, d}\ (1.8+0.6 \log_{10} w)$$

where A approximates 0.1 and w is expressed in percent.

Conclusions

1. The Bagnold formula for the threshold shear velocity appears adequate in giving a value for this term for use in calculating the rate of transport of sands normally existing on the coastal dunes of California.

2. The Bagnold formula for calculating the rate of transport appears superior to the Kawamura formula in that the value of the coefficient C in the Bagnold formula is better defined and more limited in range than the coefficient k in the Kawamura formula. That is, a value of C between 2.5 and 2.8 was adequate to define the transport of sand with a wide range of grain sizes (sands A and B) and a value of 1.5 was adequate for a sand that is nearly uniform in size (sand C). The Kawamura formula includes the threshold shear velocity, which introduces a further uncertainty in the calculations of transport rates, especially since this factor is influenced by the moisture content of the sand.

3. With a relatively high water content of the sand the windspeed necessary to initiate sand movement becomes increasingly important. This is especially true for the coastal dunes of California where the higher windspeeds, which usually occur during storms, are also accompanied by considerable rain.

Acknowledgments

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QUANTITATIVE TRACING OF LITTORAL DRIFT

[Paper No. 76]

By PER BRUUN, *engineer, Coastal Engineering Laboratory, University of Florida, Gainesville*

Abstract

The longshore littoral drift is caused by a combination of shear stresses by wave action that break loose the material from the bottom, usually pushing it in the direction of wave propagation, and by longshore currents, which help tear loose the material and transport it longshore.

In order to gain an understanding of all the littoral drift phases, it must be split up in several parts to be investigated separately. Some of these represent a combination of elements, which have to be looked into independently. Other parts may present an integration of single elements, making possible investigations in integrated form.

The Coastal Engineering Laboratory of the University of Florida is at present carrying out a combined field and laboratory research program on friction factors for longshore currents and quantity of longshore drift. The laboratory utilizes tracing technique by luminophores in the field. Wave tank experiments are carried out simultaneously. Longshore currents are passed through waters agitated by waves propagating parallel to or perpendicular to the current action.

Field Research

General

Sediment transportation is caused by a combination of shear stresses by wave action and by current action. The wave-induced shear stresses "break loose" the material from the bottom while the longshore currents transport it longshore.

Figure 1 demonstrates a typical situation

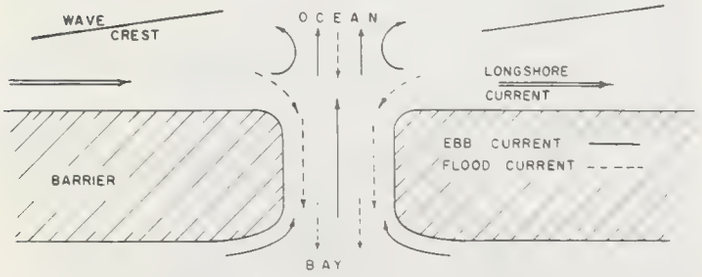

FIGURE 1. — Typical current action of barrier and inlet shore.

from a seashore like the U.S. east and gulf barrier-lined coasts. Wave action causes a longshore current that will be strongest in the breaker and uprush zones. This current will be interrupted on the unbroken barrier shore by rip currents and on the inlet broken shore by tidal inlet currents (fig. 1). This means that we are normally dealing with wave and current action perpendicular to each other. Occasionally, at inlet entrances, they run parallel to each other, either in the same or in opposite directions.

A research program on littoral drift must include the above-mentioned overall problems. The procedures described below are based on the philosophy that, although it is not possible to understand the complex problems in full unless research penetrates down in the very details (*7*), it is still possible to obtain results of scientific and practical value by considering an integrated part of the problem, including a variety of details to be looked into when the integrated problem has been investigated or analyzed to the extent this is possible.

Modes of Transport

As in rivers littoral material transport occurs partly as bedload and partly as suspended load transport. It is known that bedload transport takes place in two basically different ways — partly in a fast moving sheet layer on the surface of the bottom and partly in bulk movements as slowly migrating waves on the bottom, perhaps in a multiwave system as observed in the Mississippi River where whole systems of waves advance by an overhauling procedure (*13*). Most likely the situation on the seashore is similar to the situation in rivers as mentioned in a later paragraph.

Sheet-Layer Movement

The thickness of the sheet layer depends upon the shear stresses and lift forces exerted upon the bottom and their variation with time. Under the influence of these stresses, the sand particles travel parallel to the wave crests (longshore) when the oscillating water particles are mainly in vertical movement and they migrate perpendicular to the wave crests (and to the shore) below wave crests and troughs. The thickness of the sheet layer as observed by some researchers may correspond to the height of the ripples as long as they occur on the bottom. Meanwhile, with higher water velocities the ripples are wiped out and replaced by a flat, slightly undulated surface, which carries a considerably higher bedload movement and probably increasing thickness of the "live" surface layer. Certain field and laboratory tests on sheet-layer movement are now in progress at the Coastal Engineering Laboratory of the Uni-

versity of Florida. In the field quantitative tracing is undertaken with luminescent tracers. A brief review of the manufacturing process and the proposed scanning of samples by Teleki is given in paper No. 76A. Experiments in which fluorescent tracers are used are mentioned in references (*9*, *10*, *17*, *23*, *24*, and *25*). Examples of radioactive tracing are given in references (*18* and *20*) and in papers by L. L. McDowell and by Cummins and Ingram in this Proceedings. Kamel and Johnson (*12*) mention natural tracers.

With respect to sheet-layer movement, the following procedure is used at experiments in progress at the Fernandina Beach, northeastern Florida.

Tracer material is dumped by diver or from helicopter on the bottom on the updrift side of an 800-ft. long pier extending in the ocean to a depth of 15 to 20 ft. at mean sea level (fig. 2). Suspended load samplers as well as bedload samplers are operated from the pier at various depths. The suspended load sampler is a pump with a suction pipe having four vertical intake pipes and settling basins on the pier. The bedload sampler is also supposed to function as a

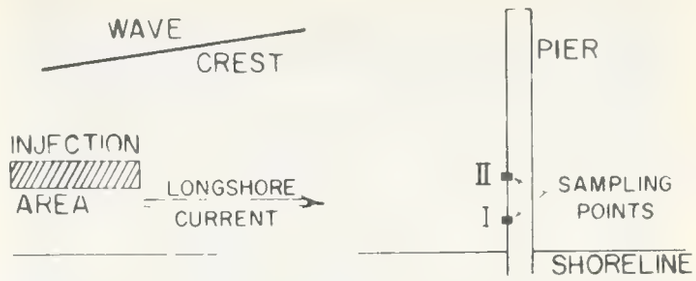

FIGURE 2. — Location of tracer material in relation to Fernandina Beach pier, northwestern Florida.

trap. It is a turtle-shaped steel and plastic dome about 3 ft. long, 2 ft. wide, 8 in. high, with remote-controlled doors at each end (fig. 3). To avoid excessive scour around the trap and its openings a heavy steel apron is attached to the bottom of the trap. The proper function of the trap is checked by the diver. Placed parallel to shore, the trap is operated for a few minutes, after which doors are closed and water jets circulate the water in the trap at the same time as a vertically slitted pipe in the middle sucks up the material accumulated in the middle for deposit in basin on the pier.

FIGURE 3. — Bedload trap.

By continuous sampling of material, the number of grains/time relationship is obtained. If n tracer grains are trapped per unit time, one has v_x=travel velocity of grains, b=thickness of sheet layer, w=width of trap opening, and k_1 a material constant

$$w \cdot b \cdot v_x = n \cdot k_1$$

$$b = \frac{nk_1}{wv_x} = k_2 \frac{n}{v_x} \text{ ; where } k_2 = \frac{k_1}{w} \qquad (1)$$

"b" indicates the thickness of the imaginary layer of coated grains and is very small. The corresponding number of noncoated grains coming from the same bottom area as the coated grains is unknown. Most grains trapped in the period immediately after the dumping of tracer grains come from the area close to the pier.

If relatively heavy concentrations are assumed, coated grains will arrive at the pier in a stream that most likely will increase rather rapidly to a maximum value and then fade out slowly (fig. 4). When point A of figure 4, indicating maximum concentration, has reached the pier, discharge of material from the injection area has its maximum value. The corresponding maximum thickness of material arriving from the injection area is

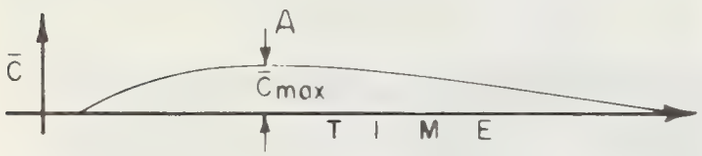

FIGURE 4. — Theoretical relationship of concentration of tracer material versus time.

$$B_{max} = \frac{N}{n_{max}} \cdot \frac{n_{max}}{V n_{max}} \cdot k_2 \qquad (2)$$

when N is the total number of grains trapped per unit time.

Next the assumption is made that the ratio $\frac{N}{n_{max}}$ can be considered as a characteristic all-time valid figure for the release of material from the injection area and is true for all the various sheet layers sliding on top of each other with different velocities.

The use of $\frac{N}{n_{max}}$ as "standard ratio" assumes that coated and noncoated grains leaving the injection area have the same fate. This means that full natural mixing has taken place, eliminating all irregularities that would tend to create fluctuations in concentrations at the sampling point. Tracer material, therefore, should be released in a certain area of the bottom parallel to shore, thereby eliminating the influence of smaller migrating sand waves. It may not be possible to eliminate the influence of the very large waves.

It still cannot be expected that maximum concentration will occur in one distinct peak of concentration. Therefore, as an example the $\frac{N}{n_{max}}$ ratio should be determined as the average of 5 maximum concentration samples:

$$\left(\frac{N}{n_{max}}\right)_{ave.} = \frac{1}{5} \frac{\overset{5 \text{ max } n \text{ values}}{\sum N/n_{max}}}{1 \text{ max } n \text{ value}}$$

The total thickness of the fast moving sheet layer up to maximum concentration may then be found by an integration:

$$B_{total} = \Sigma b = \frac{\overset{n=n_{max}=n_j}{\sum N/n_{max}}}{\underset{n=n_1}{}} k_2 \frac{n_j}{V_j} \qquad (3)$$

The total quantity of material (Q_{total}) in all sheet layers may be found by direct observation of the quantity trapped. The integral below includes fast- as well as slow-moving material.

$$Q_{total} = Q_{fast} + (Q_{slow}) = \frac{\overset{n=n_{max}=n_j}{\sum N/n_{max}}}{\underset{n=n_1}{}} k_2 \frac{n_j}{V_j} \cdot V_j + (Q_{slow}) \qquad (4)$$

The thickness and migration velocity of the slower and very slow moving bottom material may be determined by continued long-range observations beyond maximum concentration and may in the extreme include the migrating humps of material that has erosion slopes updrift and deposit slopes downdrift.

Figure 5 shows one of the results of introductory tests at Fernandina Beach pier, where a trap was used as shown in figure 3. When figures 4 and 5 are compared, this result is encouraging. To obtain reliable results, it is

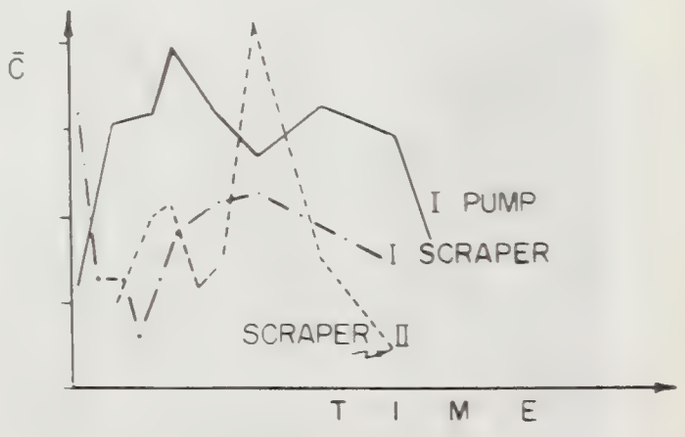

FIGURE 5. — Results of a test of concentration of tracer material versus time.

necessary to inject various color schemes in strips parallel to shore.

The N/n ratio is independent of the direction of travel of material, but the trap used to pick up material and measure quantity of longshore drift must assure that the material actually is derived from the longshore movement of sheet layer corresponding to the width of the door of the trap. Placement of a steel apron around the trap prevents erosion of material that would take place at corners, which could interfere with deposits in the trap.

The results of these experiments with bed and suspended load are only valid for the conditions under which the experiments were actually run. In order to gain information useful in practice, a great many results under a variety of conditions are needed. These results must link wave and current characteristics to sediment transportation character. Concerning the hydraulic character, efforts should be intensified to determine more detailed data, such as bottom velocities, their directions, and fluctuations and turbulence in the grand as well as the small scale. The program is therefore not in the least an attempt to determine the detailed flow patterns and velocity characteristics and its relation to the sediment transport. Velocities parallel as well as perpendicular to shore therefore are measured in front of the updrift door by instruments attached to the trap.

Bulk Movement of Sand in Migrating Sand Waves

The ocean floor does not have an even surface when considered on the grand scale. In addition to the longshore bars it is provided with undulations perpendicular to the shoreline. These undulations migrate "downstream" in the way that part of the material creeping up along (or eroded from) the updrift slope comes to rest on the downdrift slope thereby causing a downdrift movement of material. This in turn means that thickness of the surface sheet layer on the updrift side of the undulations is greater than on the downdrift side.

Such undulations have been observed in rivers. Lane and Eden (*13*) state that 350 million tons of sediment is transported annually in the lower Mississippi, mostly in suspension. Meanwhile surveys demonstrated that considerable quantities of bed material migrated along the bottom in large bars with varying heights of 5 to 10 ft. and lengths of 1,000 to 4,000 ft. and a travel velocity of 15 to 25 ft. per day, causing a material movement of 2,000 to 8,000 cu. yd. per day. This is further explained by Carey and Keller (*6*), with special reference to the friction elements involved. Similar phenomena are well known from flume experiments. On the seashore migrating sand waves moving toward the shore are mentioned by Thierry and van der Burgt (*21*) with reference to observations in the English Channel by Hook van Holland and Den Helder.

Bruun (*1*) mentions results of research on this phenomenon on the Danish North Sea coast where large undulations migrate along the shore in the direction of the littoral drift in the area of the beach profile near the shoreline. The "wave-length" seems to be 0.5 to 2 kilometer, the "wave height" (horizontal) 60 to 80 meters. Moreover, detailed depth soundings on the Danish North Sea coast proved the existence of deep-water, large, wave-shaped sand humps that migrated along the shore. The wavelength seemed to be 1.5 to 3 kilometers, the vertical height is 1 to 2 meters.

Similar experiments were carried out and are still in progress on the Atlantic shore at Fernandina Beach and at St. Augustine, Fla. At Fernandina Beach profiles were surveyed up to about 20 ft. depth at regular intervals. Figure 6 shows the results of such a survey made. It

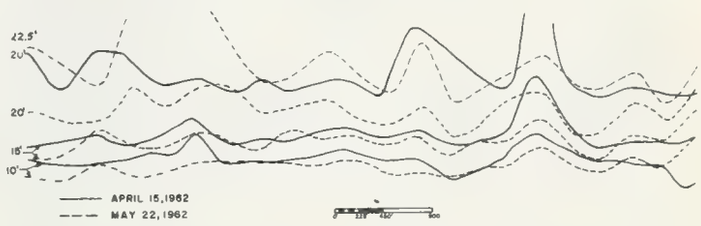

FIGURE 6. — Profiles of migrating sand waves, Fernandina Beach.

appears that the wavelength of undulations in depth contours vary from about 500 to about 1,500 ft. and that the horizontal wave height may be 1,000 ft. or more.

Laboratory Research

Transport by Tidal Flow; No Wave Action

The material transportation must follow the same physical laws as valid for unidirectional flow even if the velocity varies during the tidal half cycle. Bruun and Gerritsen (*4*) mention rational approaches in relation to tidal hydraulics. A review of "Engineering Aspects of Sediment Transport," (*2*) reports the results of research with unidirectional flow.

Transport by Wave Action

Several flume experiments have been made on transportation of bed material by wave action and on the distribution of velocities, including mass transport velocities. Russell and Osorio (*17*) found by tests in a wave flume that near the sand bottom drift velocities are as predicted by Longuet-Higgins' theory (*15*) for all waves investigated in their tests. Whether the

bottom drift was on a horizontal or a sloping bottom within certain limits of steepness, it was always in the direction of propagation of wave action. So was the direction of the drift in the surface. However, in the middle of the wave flume the drift was seaward, or against the direction of propagation of wave action.

Lhermitte (*14*), in similar experiments considered the formation of ripples and the influence of viscosity in the bottom boundary layers.

It is generally known that the friction parameter depends partly upon the bottom configuration (ripples, dunes, smooth, etc.) and partly upon the movability of the undulating boundary layer. Whether the flow is unidirectional or oscillating, it is difficult to measure water velocity and particle velocity separately, unless the material is very fine (wash load), in which case the sediment and water particle velocities are identical. If the friction factor as related to the bottom configuration and the damping influence on turbulence caused by the fine sediment particles are known, it is possible to use the Prandtl-von Karman velocity distribution law, with corrected constants for unidirectional flow (*8*). But for a coarse sand bottom it is difficult or impossible to arrive at any definite "velocity profile," because of the great irregularities in velocities. Results of model experiments on scours at sluices in Holland proved that the velocity fluctuations is the important parameter, because it gives a measure for the turbulence that is all-important for the combined shear and lift forces which are responsible for material movement.

Transport by Tidal Flow and Wave Action Running With or Against Current Action

Research is in progress in a 4 ft. deep, 6 ft. wide, wave tank (fig. 7), which also functions

FIGURE 7. — Wave tank for littoral drift experiments.

as a flume. Before wave action and current action are introduced simultaneously in the tank, friction factors as well as sediment transportation are measured for the two independent movements of flow. For larger scale conditions of oscillating water movement an oscillation tank, as described by Bruun and Gerritsen (*4, p. 107*), may be needed.

Transport by Currents Perpendicular to Inlet Flow: No Wave Action

Inman and Bowen (*11*) found, in flume experiments on combined wave and current action propagating in the same direction, that the transport rate of bed material was related to loss of wave energy for all low values of current velocity. Increases of the current velocity beyond a certain value created a complex sediment movement above the ripples, which were periodic but asymmetric; the phase-dependent phenomena then had a greater effect on the system than that of the drift velocity.

Transport by Currents Perpendicular to Inlet Flow: Wave Action

Wave action may be operating under various angles with the current activity. Inasmuch as a current running parallel to shore has to turn about 90° in order to enter the inlet, angles between 0° and 90° occur. The about 90° angle is the situation that we find with normal longshore current activity.

The relative importance of the two sediment-agitating water movements is considered. As long as the longshore current movement is predominant, it may be expected that "wave friction factors" have considerable similarity to friction factors for unidirectional flow.

The longshore current theory by Bruun (*3*), using the principle of continuity, has two approaches. One, "the rip current approach," particularly refers to moderate to lower velocity longshore currents resulting from smaller angles of incidence of wave crests. Examples based on statistical wave distribution give us reason to believe that the actual friction factor may be a combination of normal bottom friction and the hydrodynamic friction factor, depending upon the wave motion.

The other approach is called "the slope approach" and is based on inflow of water from wave breaking under a certain angle, with the shoreline giving rise to longshore "hydraulic slope" of the water table.

Examples for one and multibar profiles (*3*) give reasons to believe that this approach, using corrected experience values of the Chezy factor C, may be valid for the higher velocities resulting from larger waves that break under a rather big angle with the shoreline.

With respect to the littoral drift as bedload, wave motion will tend to increase net transport by increasing the bottom shear stresses. Furthermore, wave motion will tend to create seepage forces into and out of the bed. This causes an added boundary shear. In shallow water, the seepage force is of sufficient magnitude to decrease the effective weight and, hence, the stability of the bed material as much as 20 percent.

Figure 8 shows a cross section of the wave tank depicted in figure 7. As explained above, waves are passed down the tank through the 6-ft. wide section. At the same time a longshore

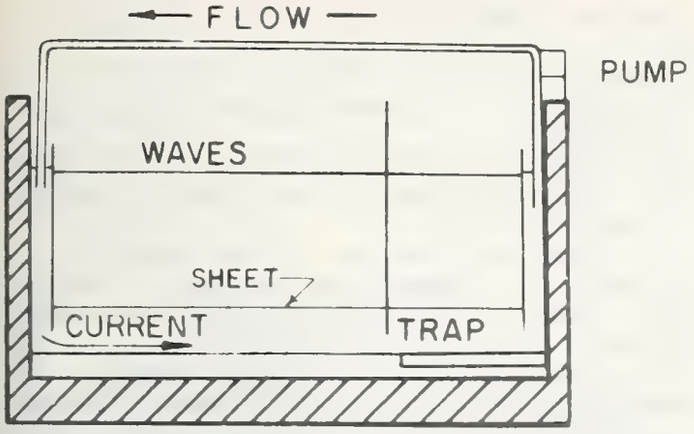

FIGURE 8. — Cross section of wave tank for littoral drift experiments.

current is run through the T-shaped end of the tank along the bottom perpendicular to the direction of wave propagation. To avoid eddy formation and dispersion of sand material from the bottom, a plastic sheet is put in about 20 centimeters above the bottom in the upper water layers of the tank. This sheet does not disturb the general motion of water particles in almost closed paths, but it hinders the free development of turbulence where it is located.

Wave and current characteristics are combined in the way that only one factor is varied while the others are kept constant. The head difference needed to keep the longshore bottom current constant varies with changes in the combined hydrodynamic and bottom roughness.

It is thereby possible to obtain data on friction as well as bedload transport parameters that range over an area from predominant unidirectional flow to predominant wave motion. In case of predominant unidirectional flow, the orbital velocities near the bottom are small compared to the longshore current velocities. For predominant wave motion, the shear stresses exerted on the bottom by the oscillating water determine the friction and transport parameters. Bedload transport is determined by bed traps installed as shown in figure 8. Its relation to wave and current conditions is recorded.

Most likely the results may indicate the following dependencies in friction as well as sediment transport parameters:

Predominant wave motion — *Wave and current action* — *Predominant longshore current*

$$T_w^{n_1} \cdot f(T_w)^{n_2} \quad (T_w \cdot T_{long})^{\frac{n_1}{2}} \cdot f(T_w)^{\frac{n_2}{2}} \cdot f(T_{long})^{\frac{n_2}{2}} \quad T_{long}^{n_1} \cdot f(T_{long})^{n_2}$$

in which T_w= the wave shear stresses and T_{long} =the longshore current shear stresses.

Littoral Drift at Tidal Inlets

General

One of the main results of research on tidal inlets mentioned in reference (*4*) is expressed in the relation

$$A = \frac{Q_m}{C\sqrt{\frac{T_s}{\rho g}}} \qquad (5)$$

in which A=cross sectional area
Q_m=maximum discharge
T_s =determining shear stress for bottom stability
C =Chezy's friction factor
ρ =density of water
g =the acceleration of gravity.

The "stability shear stress" (T_s) proved to be an important factor for gorge stability. Bruun and Gerritson (*4*) gave the data for the "large scale tendency" as follows:

T_s with heavy sediment load=0.50 kg./m².
T_s with medium sediment load=0.45 kg./m².
T_s with light sediment load=0.35 kg./m².

This refers to inlets with sand bottom 0.2 to 0.4 mm. diameter.

Furthermore it was found that the Ω/M ratio (Ω=tidal prism in cubic yards per half tidal cycle, M= net (predominant) littoral drift in cubic yards per year) seems to be useful for description of the actual "degree of stability" of the inlet.

It was concluded that ratios of $\Omega/M<100$ should be avoided and that ratios $\Omega/M>200$ are desirable for "stable inlets." The corresponding Q_m/M value (Q_m=peak discharge per second) should be >0.01 to obtain "stability." If the $M/Q_m=r$ ratio was considered, it was found that this ratio is useful for the inlet's ability to bypass sand (*5*):

If $r>200$ to 300, bypassing takes place on a bar.
If $r<10$ to 20, bypassing by tidal flow takes place.

As a result of this finding, initiation of a research program, as described above, comprising investigations of friction and shear stress parameters under combined wave and current action, was suggested.

Furthermore, model studies were suggested to look into a number of pertinent tidal hydraulics and littoral drift factors and ratios between factors.

Research Based on Existing Field Data

Although the relative magnitude of tidal currents and rate of littoral transport expressed by the ratios Q_m/M or Ω/M plays an important role for inlet cross sectional stability, the ability of the inlet to transport solid matter by flow-induced shear stresses (rather than by the

"tidal prism" or by the "maximum discharge") must be important.

If neither scour nor accretion occurs in the inlet, the following must be valid:

(1) The littoral drift material is transferred by combined action of inlet currents, waves, and longshore currents. The total amount of sand that is handled in a certain time period is not the net littoral transport but equals the total amount of littoral transport carried to the inlet in that time, written $\Sigma|M|$, if the transport from both sides of the inlet is considered.

(2) The amount of littoral drift entering the inlet from both sides in a certain time equals the amount of net littoral drift (V_s) that is flushed out of the inlet channel by tidal currents during the same time period. One has:

$$V_s = p\Sigma|M| \qquad (6)$$

where $100p$ is the percentage of $\Sigma|M|$ that is handled by tidal currents and $\Sigma|M|$ is the total longshore littoral transport during one tidal cycle.

With respect to equation 6, it should be borne in mind that part of the bedload in the inlet moves back and forth as a "rolling carpet" (*4*) without being delivered to the downdrift side of the inlet during the ebbtides. From this it follows that the maximum value of p, occurring in case of highly predominant tidal flow bypassing, may easily exceed one. Further, with very low values of p, the inlet cannot survive since it is choked.

The more predominant the tidal flow bypassing is (high p-value), the more stable is the inlet in location and cross section.

The ratio $p = \frac{V_s}{\Sigma|M|}$ has been calculated for various inlets in order to check the above-mentioned considerations.

In most cases M_{net} and not $\Sigma|M|$ is known, although only in approximation. The assumption therefore was made that the littoral transport in one direction was 1.5 times the littoral transport in the opposite direction, which means that $\Sigma|M| = 5\ M_{net}$.

V_s was calculated, using Kalinske's formula for bedload transport and is indicated in table 1. Other bedload formulas valid for the same range as Kalinske's expression will give a similar result. The velocity range in question is confined to 3 to 4 ft./sec., for which reason the "proper choice" of bedload formula is not very essential.

TABLE 1.—*Some V_s, M_{net} and p values*

Inlet	V_s per half tidal cycle	M_{net} per year	p (ratio)
	Cubic yards	*10^6 cu. yd.*	
Oregon, N.C.	3,200	1.0	0.4
Calcasieu, La.	1,800	.1	1.3
Port Aransas, Tex.	2,100	.1	1.4
Mission Bay, Calif.	750	.1	1.0
St. Johns, Fla.	1,610	.3	.8
St. Augustine, Fla.	1,000	.3	.5
Big Pass, Fla.	540	.1	.7
Ponce de Leon, Fla.	1,040	.5	.3
Gasparilla, Fla.	1,200	.1	1.5

The p values obtained vary between 0.3 and 1.5; and seem to describe the actual inlet stability fairly well.

A considerable number of field data (*4*) are available that furnish information of importance for the understanding of "cross-sectional stability" characterized by the Ω/M and Q_m/M ratios as well as the "material transfer stability" described by the M/Q_m ratio. The $p = \frac{V_s}{\Sigma|M|}$ ratio is a further development of the above mentioned ratios. The interest is now particularly concentrated in gaining more knowledge about this factor and its variations, because with this knowledge many other problems may be understood. Research in progress include:

1. Investigations on the importance of the p value in general and in comparison with field data.
2. Investigations of the importance of the actual value of p for inlet stability as a whole.
3. Investigations on the importance of fluctuations of p. V_s as well as $\Sigma|M|$ varies in the field. Such variations and their reflections in "degree of inlet stability" are of utmost importance for the description of actual inlet stability as it presents itself during seasons with varying flow and sediment load. For this reason a check on the sensitivity of the model, with reference to slight changes of the relative magnitude of sediment transport by inlet and longshore currents, is important and will be handled by repeating each test for a certain value of $p = \frac{V_s}{\Sigma|M|}$ with comparatively small changes in p. The conclusion may be drawn after a few such tests that the observed phenomena are significant for a particular value of p if, in tests run with essentially different values of p, significant differences are observed and if the tests with about the same value of p did not show significant differences.

For such experiments reliable model scales are essential.

Model Laws

The following section is an abstract of a paper by Bruun and Battjes presented at the International Association of Hydraulic Research in London, September 1963.

Similarity in Bedload Transport in Uniform Flow:

Tidal Current By-Passing $(\frac{\Omega}{M} > 200)$

Similarity in bedload transport requires similarity in initiation as well as in rate of transport (*10*). Shields' (*19*) investigations indicated that

$$\frac{\tau_c}{(\gamma_s - \gamma) D} = f_1 \left(\frac{v_* D}{\nu}\right) \quad (7)$$

and

$$\frac{k}{D} = f_2 \left(\frac{v_* D}{\nu}\right) \quad (8)$$

As to the rate of transport, most of the published formulas can be written in the dimensionless form

$$\frac{q_s}{\nu} = \frac{v_* D}{\nu} \cdot f_3 \left(\frac{\tau}{\tau_c}\right) \quad (9)$$

where

$$\tau = \rho\, v_*^2 = \rho g h I \quad (10)$$

From this it follows that similarity in bedload transport can be obtained if

$$\lambda \left(\frac{v_* D}{\nu}\right) = 1 \quad (11)$$

and if

$$\lambda_\tau = \lambda_{\tau_c} = \lambda(\gamma_s - \gamma) \cdot \lambda_D \quad (12)$$

The scale λ is defined by

$$\lambda_n = \frac{n_{\text{prototype}}}{n_{\text{model}}} \quad (13)$$

We assume that

$$\lambda_\nu = \lambda_\rho = \lambda_g = 1 \quad (14)$$

From equations 9, 11, 12, and 14 it follows that

$$\lambda_{q_s} = 1 \quad (15)$$

If the Froude number of the flow

$$\mathrm{Fr} = \frac{v}{\sqrt{gh}} = \frac{v}{\sqrt{ghI}} \sqrt{I} = \frac{C}{\sqrt{g}} \cdot \sqrt{I} \quad (16)$$

should be identical in model and prototype, we have

$$\lambda_I^{1/2}\, \lambda_C = 1 \quad (17)$$

where C is the Chezy coefficient that in fully developed turbulent flow depends on the relative roughness:

$$C = 5.75 \sqrt{g} \log 12 \frac{h}{k} = 18 \log 12 \frac{h}{k}\ m^{1/2}/\text{sec.} \quad (18)$$

From equations 8 and 11 it follows that

$$\lambda_k = \lambda_D \quad (19)$$

Substitution of equation 10 into equation 11 and of 18 and 19 into equation 17 yields

$$\lambda_h^{1/2}\, \lambda_I^{1/2}\, \lambda_D = 1 \quad (20)$$

$$\lambda_h\, \lambda_I = \lambda(\gamma_s - \gamma)\, \lambda_D \quad (21)$$

$$\lambda_I^{1/2} = 1 + \frac{18}{C_p} \log \frac{\lambda_D}{\lambda_h} \quad (22)$$

Equations 20, 21, and 22 give three relationships between the four scales λ_h, λ_I, λ_D, $\lambda(\gamma_s - \gamma)$, of which one can be chosen freely and the others solved. The scale λ_D cannot equal λ_h, since in that case all the scales equal unity. Therefore, $\lambda_k = \lambda_D \neq \lambda_h$ and then as follows from equations 17 and 18

$$\lambda_C = \lambda_I^{-1/2} \neq 1 \quad (23)$$

so that a distorted model is necessary.

The above theory can be extended to the case that artificial roughness elements (pegs) are needed. Obviously, equation 10 then is no more valid if τ represents the shear stress on the bed. In order to find the actual τ, the characteristics of the artificial roughness elements have to be considered.

The condition of zero acceleration of a column of water of unit length and width and height h in which n pegs are placed with drag coefficient c_D and projected area F is

$$\rho g h I = \frac{\rho g}{C_o^2} v^2 + n c_D F \cdot \tfrac{1}{2} \rho v^2 \quad (24)$$

where C_o is the Chezy coefficient for the channel without pegs.

Inserting

$$v = C_m \sqrt{hI} \quad (25)$$

into equation 24 gives

$$C_m = C_o \sqrt{\frac{2g}{2g + C_o^2\, n c_D F}} = \frac{C_o}{\alpha} \quad (26)$$

so that equation 22 is replaced by

$$\alpha\, \lambda_I^{1/2} = 1 + \frac{18}{C_p} \log \frac{\lambda_D}{\lambda_h} \quad (27)$$

Rewriting equation 24 and substituting equation 26 gives

$$\frac{\rho g}{C_o^2} v^2 = \tau = \frac{\rho g h I}{1 + \tfrac{1}{2} C_o^2\, n c_D F / g} = \frac{1}{\alpha^2} \rho g h I \quad (28)$$

from which it follows that

$$\lambda_\nu = \lambda_{v_*^2} = \alpha^2\, \lambda_h\, \lambda_I \quad (29)$$

Substitution of equation 29 into equations 11 and 12 gives

$$\alpha \lambda_h^{1/2}\, \lambda_I^{1/2}\, \lambda_D = 1 \quad (30)$$

$$\alpha^2\, \lambda_h\, \lambda_I = \lambda(\gamma_s - \gamma)\, \lambda_D \quad (31)$$

The pegs should be placed in such a way that they interfere as little as possible with the velocities near the bottom. The Karman vortices should not influence the movement of the bottom material. This may be done by putting the pegs on a thin rigid wire; the best shape and disposition of the pegs as well as their drag coefficient can be investigated in a separate flume study.

Similarity in Bedload Transport by Waves:

Bar Transfer $(\frac{\Omega}{M} < 100)$

The same criteria as above apply, viz., equations 11 and 12 are valid. For τ the expression derived from the theory of the laminar boundary layer due to wave action is used (*22*):

$$\tau_{max} = \nu^{1/2} \rho v_{max} T^{-1/2} \quad (32)$$

where

$$v_{max} = \frac{\pi H}{T} \sinh^{-1} \frac{2\pi h}{L} \quad (33)$$

Similarity in wave pattern is a demand also and may be obtained by making

$$\lambda_L = \lambda_h \quad (34)$$

Substitution of equations 32, 33, and 34 into equations 11 and 12 yields

$$\lambda(\gamma_s - \gamma)\lambda_D^3 = 1 \quad (35)$$

and

$$\lambda_H^4 = \left(\gamma_{(\gamma_s - \gamma)}\right)^{8/3} \cdot \lambda_h^3 \quad (36)$$

Equation 35 follows from equations 11, 12, and 14 regardless of the value of τ and is therefore equally valid for current- or wave-induced bedload transport similarity.

If similarity in the position of the breakerline is required, we get:

$$\lambda_H = \lambda_h = \left(\lambda_{(\gamma_s - \gamma)}\right)^{8/3} \quad (37)$$

since the point of breaking in shallow water is mainly determined by the ratio $\frac{H}{h}$.

Similarity in Suspended Load Transport Perpendicular on the Shoreline by Waves

A necessary although not sufficient condition is

$$\frac{\text{terminal fall velocity}}{\text{mass transportation velocity}} = \frac{\text{vertical distance}}{\text{horizontal distance}} = \lambda_I \quad (38)$$

From equation 38 the following relationship is obtained by the use of Longuet–Higgins' theory on mass transport and equations 35 and 36:

$$\lambda_I = \lambda(\gamma_s - \gamma)^{-1} \quad (39)$$

Symbols for Model Laws

C, C_o, C_m ..... Chezy coefficients ($L^{1/2} T^{-1}$)
C_D ..... Drag coefficient
D ..... Grain diameter (L)
F ..... Area of a peg perpendicular to direction of flow (L^2)
f_1, f_2, f_3 ..... Functions
g ..... Acceleration of gravity (LT^{-2})
h ..... Depth of flow (L)
H ..... Wave height (crest to trough) (L)
I ..... Slope
k ..... Apparent roughness (L)
L ..... Wave length (L)
m ..... Subscript "model"
M ..... Average net littoral transport per year ($L^3 T^{-1}$)
n ..... Number of pegs per unit area (L^{-2})
p ..... Subscript "prototype"
q_s ..... Rate of bedload transport per unit width by volume ($L^2 T^{-1}$)
T ..... Wave period (T)
v_* ..... Shear velocity (LT^{-1})
v ..... Average velocity (LT^{-1})
v_{max} ..... Maximum orbital velocity near the bottom (LT^{-1})
a ..... Coefficient
γ ..... Specific weight of fluid ($ML^{-2}T^{-2}$)
γ_s ..... Specific weight of sediment ($ML^{-2}T^{-2}$)
λ ..... Scale ratio, prototype over model value
ν ..... Kinematic viscosity of fluid (L^2T^{-1})
ρ ..... Specific density of fluid (ML^{-3})
τ ..... Shear stress on the bottom ($ML^{-1}T^{-2}$)
τ_c ..... Critical shear stress for the initiation of bedload movement ($ML^{-1}T^{-2}$)
Ω ..... Tidal prism (L^3)

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A SUMMARY OF THE PRODUCTION AND SCANNING OF FLUORESCENT TRACERS

[Paper No. 76A]

By P. G. TELEKI, *assistant in research, College of Engineering, University of Florida, Gainesville*

Tracer Materials

Because of the large areas involved in tests in the field and rapid and continuous spreading of materials observed in Russian and British experiments (*1, 4, 11, 16, 20*), large quantities of tracer materials are needed and it is anticipated that the measuring unit for this material may be in terms of tons. This does not necessarily mean that several tons of tracer sand will have to be coated. According to British experience, it should be possible to dilute the coated sand 50 to 100 times with noncoated grains before release (*4*).

When manufacturing fluorescent tracers, one must take into account that the material obtained cannot differ in its physical properties from the original beach sand which it represents (*10*). Hence, it is imperative for the finished product to have the same density, hardness, and actual dimensions as the original. Shape, roundness, and sphericity values should not be altered. Requirements include that no chemical reactions would take place between tracer and seawater, that it would withstand considerable abrasion, be distinguishable and detectable in minute quantities, and its intensity of color would be subject to no change within a required period of time (*5*).

Fluorescent organic dyes, those that are water-soluble, oxidize on exposure. However, these dyes are readily available, and properly used they provide a simple means of detection. Easily dissolving in resins and set by solution-type hardeners, they increase the radius of a grain by only 300 millimicrons.

The following is a description of a procedure used by the Coastal Engineering Laboratory of the University of Florida. Quantities are approximate.

(1) 1 to 4 g. of dye is dissolved in 2.5 liters of catalyst (20 percent solution).
(2) The solution is added to 25 lb. of resin.
(3) The mixing of ingredients is carried out until color uniformity appears.
(4) The mixture is added to 200 lb. of dried sieved sand in a concrete mixer, wherein coating of the grains takes place.
(5) The wet tracer is placed in the sun on 6-mil-size polyethylene sheeting and spread thin for drying (2 to 3 hr.) or in an oven at up to 250° F. (1 hr.).
(6) Upon setting, the material consolidates into an aggregate that is quite friable. This is fed into a granulator which "delumps" it. The procedure takes about 15 min. for 200 lb. of tracer.

(7) The material is sieved back to its original grain-size distribution by using a sieving machine with large (22-in. diameter) sieves (¾ hr. for 200 lb. of sand).

Resins of both solution and suspension types may be used, depending on the dye to be incorporated. In turn, pigments, water-soluble, oil-soluble, or alcohol-soluble dyes can be used with proper matching of the incorporating plastic. Such combination of dyes, resins, and hardeners will give an almost unlimited range of colors, and with some experimentation one may achieve different values for tracer durability (e.g., 2 weeks, 3 months, 1 year). Visually, it is not possible to distinguish more than a few basic colors within a sample; however, with the utilization of sensitive electronic equipment, subtle shades of color may well be detected and recognized when based on spectrophotometric analyses.

With respect to dyes, resins, and hardeners, the following experience has been developed.

Dyes

Organic dyes that fluoresce in liquid solution (No. 1) (such as H_2O, ethanol, MEK, Xylene, Toluene, etc.) are best in producing good effects. Those not fluorescing in solution (No. 2) and/or those not fluorescing at all (No. 3) can be utilized in conjunction with number 1.

For example:

No. 1. Rhodamine B — fluoresces in aqueous solution (red).

No. 2. Auramine 00 — nonfluorescent in solution, but fluoresces in plastic (yellow).

No. 3. Victoria Blue B — nonfluorescent dye (blue).

No. 1 + No. 2. Rhodamine B and Auramine 00 — intense fluorescence (orange).

No. 1 + No. 3. Rhodamine B and Victoria Blue B — intense fluorescence (purple).

Pigment-type coloring often causes the problem of nonuniform coating of the grains. This depends on the amount of original resin containing the pigments in pulverized form. The use of pigments most often only accentuates the problem involved in attempting to retain the original characteristic of sand grains.

Resins

One of the most readily available and relatively inexpensive resin for coating is ureaformaldehyde. Melamine and epoxy resins can also be used, and methylmethacrylate will also provide good results. Whereas clearness, viscosity, and solid content factors make urea-type resins desirable for use, these plastics may rapidly decompose if not set with catalysts within a short time; therefore, they are hard to store. Deterioration can be delayed with controlled storing temperatures and humidity. The resistance to wear and to chemical decomposition of urea-formaldehydes is excellent in catalyzed form, as established by simulated abrasion tests.

Hardeners

An inexpensive catalyst is available on the commercial market in the form of ammonium chloride (NaH_4Cl) for use with urea-type resins. However, common acids will often suffice, such as hydrochloric (HCl) or formic acid (HCOOH).

For excitation of fluorescence, both long- and short-wave ultraviolet lamps are used at the Coastal Engineering Laboratory, Gainesville. The Hanovia-type fluorescent lamps develop short-wave ultraviolet rays in the region of 2537 Å. and long-wave rays of 3650 Å. Long-wave lamps see far more use, since fluorescent tracers have proved less expensive than phosphorescent tracers, the latter requiring detection by short-wave excitation.

Scanning of Tracers

Scanning (*6*, *12*) is a method used for detection, the differentiation of size and color, and counting of particles, and it is an important procedure associated with any tracing technique of a quantitative or statistical scale.

As long as concentrations are normal (e.g., value of 10^{-4}), a simple visual counting procedure can be used as already applied in the USSR and in England (*2*, *3*, *7*, *10*, *15*, *16*, *19*). The visual method does not necessarily mean that each single grain has to be counted, but rather that the intensity of a sample is compared with another "calibrated" sample of known content of grains (*11*, *15*, *16*). By taking subsamples out of the sample and handling the result by statistical methods, much larger actual full samples may be analyzed visually. This method will, needless to say, become inadequate as soon as bulk quantities with very high or very low concentrations are involved, making these statistical methods uncertain or impossible.

The counting method first used and still in use is visual counting by use of a luminoscope (*1*, *2*, *3*, *8*, *19*)[1][2][3] to which samples of sand are subjected under ultraviolet rays, converted from sunlight or supplied by ultraviolet lamps (*13*). This method can only handle very modest-size quantities, even if the sample in question, as mentioned, is to be interpreted as a subsample out of the actual sample taken.

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[1] Also by personal communication with P. Slomianko, Gdansk, Poland.

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Other methods able to handle the bulk quantities needed for tracing in the open ocean are under development. Russian scientists, at this time, favor a method by which the luminophore is dissolved and the actual concentration determined by chemical analysis using a fluorometer.[4] Other methods based on photocells have been suggested and are under development (*17*). Some are based on measuring the intensity of visible light reflected from the fluorescent tracers, where correlation of intensities is made with the actual number of grains in the sample (*18*).[5] Inasmuch as 1 gram of average beach sand may include 10,000 to 80,000 single grains, it is not an easy problem to count heavy concentrations. In most practical cases one is faced with the possibility of a very low concentration, which may necessitate scanning of bulk quantities of hundreds of pounds or a number of 10^{10} grains.

This requires an arrangement that is capable of handling such quantities mechanically, as well as electronically, in a fast but still fully reliable way, whether a single color or a whole series of colors are to be dealt with.

After the basic ideas had been developed at the Coastal Engineering Laboratory for the requirements and general plan of an automatic scanner for dealing with fluorescent tracers, a proposal was submitted by an electronic company for building such an instrument. In essence, the scanner as designed would complete a size analysis of the sample of beach sand and would differentiate several colors at the same time. The rate of scanning would occur at 20,000 particles per second.

The instrument when constructed will consist of —

(1) A mechanical assembly consisting of two parts: a hopper to which the sample is subjected and a Vycor glass tube in which the sand falls from the hopper at a variably controllable rate utilizing free fall.

(2) An assembly of electronic circuitry. The sample falling in the tube passes in front of several optical channels enclosing photomultiplier cells (each cell in operation accounting for one of the many given colors in the sample), whereupon the photocell responds whenever a grain with the proper color crosses the light path of the optical channel. By using a method of fixed duration pulse-signal transmitting, a counter registers the signal (hence the colored grain) digitally or graphically after the signal passed through an amplifier, a pulse shaper, and a relay driver. Each photocell excludes all but one color as set by a combination of primary and secondary interference filters incorporated into the optical channels. The dimension of the output signal transmitted by the cell to the counter mechanism registers the size of the coated sand particles simultaneously with the fixation of color. Within the counter the tabulation of color, size, and quantity occurs at the rapid rate of 1.8 kg./hr.

The theory of the scanner is basically set in the following equation. The power entering the photocell is

$$P = I\,\sigma\,a\,b\,Q\,\frac{1}{4\pi R^2}A\,c\,d\,H_v$$

where

I = Intensity of incident beam in photons
σ = Area of sand grain
a = Fraction of grain covered by fluorescent molecules
b = Fraction of irradiated fluorescent molecules which absorb incident photons
Q = Quantum efficiency of fluorescent process
R = Distance from sand grain to lens of optical system
A = Area of lens
c = Filter factor
d = Transmission factor for optical system
H_v = Energy of an emitted photon.

As this is known in terms of fluorescence, excitation of this phenomenon occurs in the ultraviolet region of the spectrum and light transmission occurs in the visible region (*14*). Since the latter is a relatively short segment of the spectral scale, all available fluorescent colors cannot be utilized for fear of misinterpretation of the overlapping curves. For the same reason, the eye cannot distinguish but a few colors when many are mixed together. In terms of differentiation the electronic scanner, using combinations of interference filters and relying upon peak outputs of the tracer colors, is able to increase the number of employable colors for any one experiment by a factor of approximately 2.5 (10 to 12 for a scanner as compared to 4 to 5 for visual means). This alone is an important reason for using such an instrument, aside from its automatic bulk quantity counting feature.

[4] Personal communication from V. P. Zenkovitch, Moscow, U.S.S.R.

[5] See footnote 3.

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DESIGN AND CONSTRUCTION OF RUBBLE-MOUND SEAWALLS AT SANTA CRUZ, CALIF.

[Paper No. 77]

By ORVILLE T. MAGOON, *coastal engineer, Navigation and Shoreline Planning Section, Engineering Division, U.S. Army Engineer District, San Francisco*

Abstract

This paper presents information on the design and construction of rubble-mound seawalls along West Cliff Drive at Santa Cruz, Calif., to illustrate a means of arresting major erosion by the ocean of a soft rock cliff formation. Offshore of this location, relatively high deep-water waves produce breakers along the bluffs. Based on the existing depth of water and the anticipated scour, a 9.5-foot design wave was calculated, which required an armor stone size of approximately 5 tons and a slope of 1 vertical on 1½ horizontal. As of November 1962, 19 units of seawall ranging from 30 to 700 feet in length have been constructed. Observation of the seawalls since construction indicates that stone size is adequate but that undesirable settlement has occurred. This settlement could probably be eliminated by placing armor stone on bedrock or below the limit of beach scour in a previously excavated trench during initial construction. Based on a recent field inspection, the seawalls appear to be serving their intended purposes.

Synopsis

This paper presents information on the design and construction of rubble-mound seawalls to illustrate a means of arresting major erosion of a soft rock cliff formation by ocean waves and currents. Prior to construction of the seawalls, considerable public and private property and about two blocks of one of the major scenic drives in the City of Santa Cruz had been progressively undermined and eroded. Further erosion of critical areas has been prevented by

construction of the seawalls, and several areas previously completely eroded have been reclaimed.

The City of Santa Cruz is located in Santa Cruz County, in the central portion of the coast of California about 65 miles south of San Francisco. Although the 1960 population of Santa Cruz was only about 26,000, Santa Cruz serves as one of the major recreational areas for the San Francisco Bay area, which had a 1960 population of about 3.5 million. During the summer months the population of the city increases fourfold or more.

One of the major visitor attractions of the city of Santa Cruz is West Cliff Drive, which extends westerly from the main beach recreational area about 2½ miles to the city limits at Natural Bridges State Park. This drive parallels the shore and provides numerous parking areas and vantage points overlooking Monterey Bay. The shore along West Cliff Drive consists of irregular eroding cliffs of soft shale, locally known as Monterey shale, averaging about 35 feet in height. No protective beaches are found in this reach. Wave erosion has caused extensive recession of the sea cliffs facing the ocean at a rate of about 1 foot annually. Severe erosion has occurred at three separate sections of the drive. In 1961, erosion caused the closure of one 2-block section of the drive and required rerouting of traffic. In addition to these three sections, a number of separate segments of shore have been undercut and large caverns exist under the road.

Along West Cliff Drive, the only method of protection considered feasible is the construction of rubble-mound seawalls. The irregular alinement and rocky nature of this reach of coast would make placement of beach fill to obtain a stabilized protective beach economically infeasible. This section of coast contains a number of projecting points and minor headlands that serve as groins. However, their projections do not impound sufficient sand to protect the bluffs from erosion. This indicates that the use of groins in conjunction with beach fill, or alone, would not correct the problem condition.

The tides at Santa Cruz are typical of those along the central California coast, and have a diurnal inequality; the mean diurnal range being 5.3 feet. The maximum tide is about 8 feet above mean lower low water (m.l.l.w.), the datum for the Pacific Coast of the United States. Characteristic deepwater waves have significant heights ranging from 2 to more than 20 feet and periods ranging from 4 to 20 seconds. The mean height and period are 4 feet and 13 seconds, respectively. The prevailing direction of wave approach is from the northwest. Winter storms with wave heights of 10 feet or more usually approach the coast from the southwest quadrant. From the data available during the design studies, the maximum deepwater significant wave height (crest to trough) was estimated to be 25 feet for a 9-second wave from the southwest. Data computed subsequent to the studies indicate that deepwater waves with a significant wave height of 33 feet and a 13-second period from the west occurred during a storm in February 1960.

In the planning and the structural design of the seawalls, use was made of the criteria contained in Technical Report 4 entitled "Shore Protection Planning and Design," published in June 1954 by the Beach Erosion Board, Corps of Engineers, and later revised and reissued in 1961. In the design of the seawalls considered, the height of the maximum wave that would break on the structure was determined from the relation, H_b (in feet) equals d_b (in feet) /1.3, where H_b is the breaker height and d_b is the water depth at the point of breaking. For design purposes, stillwater level was estimated to be 8 feet above m.l.l.w.

In determining the size of face or capstone required, it was estimated that after settlement at the toe, the slope of the seaward face of the wall would be 1 vertical on 1½ horizontal. As the depth of scour at the toe of the walls would be limited by bedrock to about –4.5 feet m.l.l.w., the maximum depth of water at the seawall toe would be about 8.0 + 4.5, or 12.5 feet. The design wave height would be 12.5 feet/1.3, or about 9.5 feet. If this design wave and a specific gravity of stone of 2.65 and slopes of 1 on 1½ are used, the required weight of face stone was found to be about 5 tons. (The required stone weight would be about 4 tons, if the 1961 revision of Technical Report No. 4 is used.) As occasional wave overtopping would produce no adverse effect, the height of the crest of the structure was adopted as one wave height above stillwater level, resulting in a crest elevation of 9.5 + 8.0 or + 17.5 feet m.l.l.w. To reduce settlement at the toe of the seawalls, a bedding layer of stone was specified when the depth of sand on the shore exceeded 1 foot. The design section is shown in figure 1.

As of November 1962, 19 units of seawall, ranging from 30 feet to 700 feet in length, have been constructed. One of the units completed in 1961 is shown on figure 2. Observation of the seawalls since construction indicates that the stone size is adequate. However, it has been noted that settlement has occurred only near the toe of the structure, thus exposing a strip of core material that has required the placement of a relatively small amount of additional stone. If found to be economical, this additional stone

placement could probably be eliminated by placing armor stone on bedrock in a previously excavated trench during initial construction. This construction procedure was used at one location in the same project where working conditions permitted construction equipment to work directly on a narrow beach or bench. In most instances, construction equipment was found to work at the top of the cliffs.

Based on a recent field inspection, the seawalls appear to be serving their intended purposes. The 2-block section of West Cliff Drive that had been eroded has been reclaimed and is again open to traffic. It is concluded that the design and construction described is a satisfactory method of protection for the shore at West Cliff Drive.

Acknowledgments

Material presented in this paper was obtained from House Document 179, 85th Congress, 1st Session, entitled "Santa Cruz County, California, Beach Erosion Control Study," and unpublished data on file in the Engineering Division of the U.S. Army Engineer District, San Francisco, Corps of Engineers.

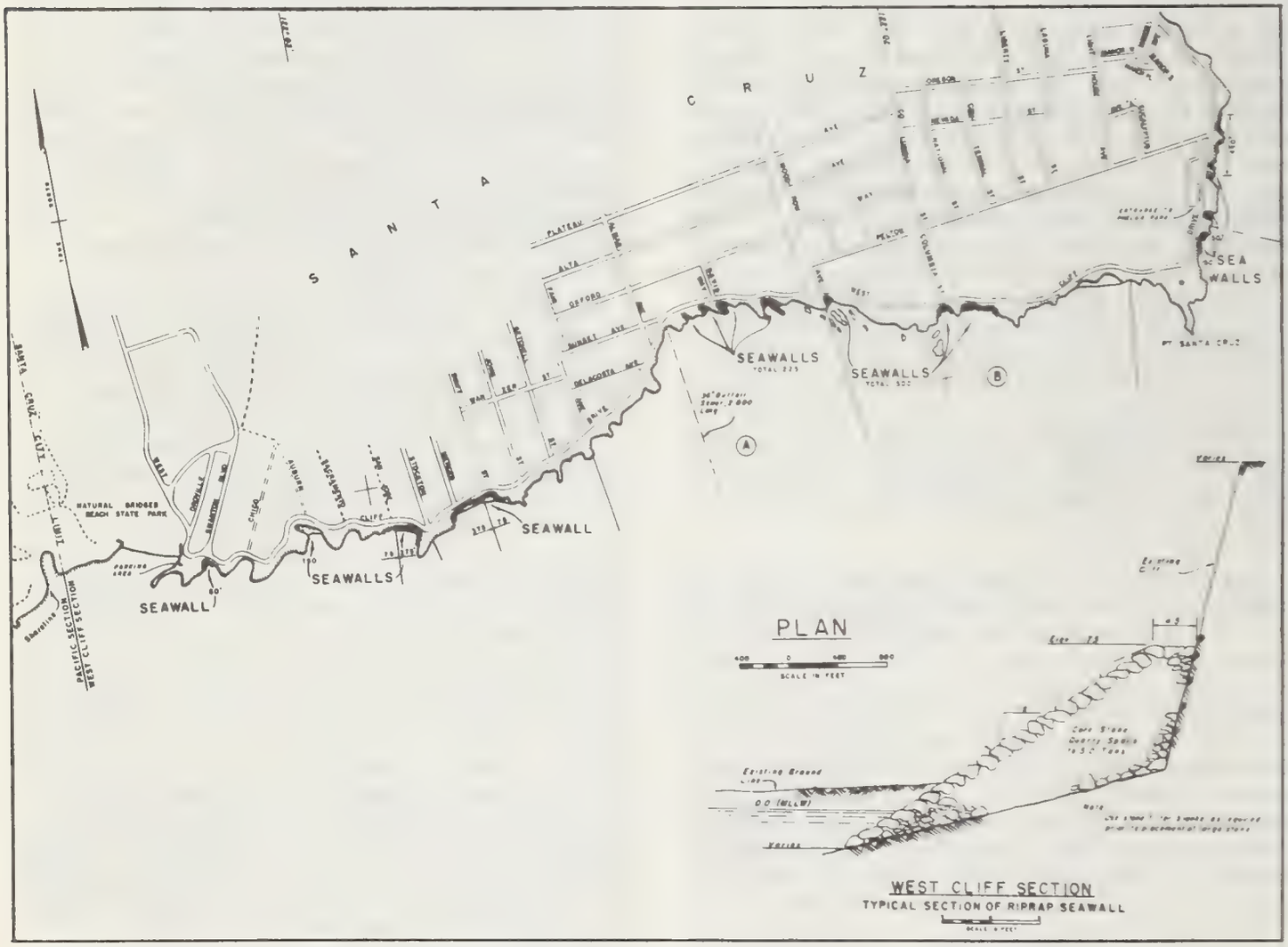


FIGURE 1. — Location of seawalls and typical seawall cross section.

FIGURE 2. — Typical seawall unit completed in 1961.

DISCUSSION — SYMPOSIUM 3
Sedimentation in Estuaries, Harbors, and Coastal Areas

Moderator: J. W. Johnson, University of California

Panel: H. A. Einstein, University of California
H. B. Simmons, CE
J. M. Caldwell, Beach Erosion Board
P. Bruun, University of Florida

MR. JOHNSON:

There are a few questions that have been submitted by those in attendance. Before proceeding with the submission of these questions to our panel, however, I am asking each panel member to describe briefly certain research that they now have in progress. Such descriptions will provide those assembled here this morning with information that may not be available in published form for a year or more in the future. I am therefore asking each panel member to present his discussion in turn.

DR. EINSTEIN:

I will not give any introduction, because we have very little time left. The work I want to refer to is connected with the information needed in making any kind of calculation or model experiments pertaining to tidal flows. As you know, we're not only interested in the motion of the sediment when we do such a thing but we also have to duplicate properly the friction conditions. In choosing the proper friction factors on mud deposits you look in vain through the entire literature to find them. Nobody has

ever been able to measure decently these values as far as we could find out, and we definitely were not able to do it in our flume experiments, because the slopes are on the order of a foot to the mile and for that a standard flume is not quite long enough. So we came to the conclusion the only way to do that is to build a very long flume and make experiments in that.

The reason why the slopes are so small is that these deposits will not stand any high flow velocities. For that reason one has to operate with velocities on the order of one foot per second. The flume that we're using is a thousand feet long, and we're using this flume, as I said, just for the purpose of determining these friction factors. At the moment we're learning how to handle this monster. It takes a long time to achieve equilibrium; one must be very careful not to induce any wave action as that would make the measurements difficult. We hope to get some results in about a year. The work is supported by the National Science Foundation.

One more remark to those who are interested in dredging. Lately, we have been called in to help with the design of some dredges that were supposed to handle very large amounts of sands and to pump these up from rather large depths up to 200 feet. In that case, you need a booster pump at the end of the ladder. A jet pump was selected for the booster. You know that a jet pump has the advantage that there are no movable parts in connection with the flow. For that reason minimum wear was expected. We found, and this is the reason why I mention it, that there is an unexpected benefit due to the use of these pumps. Since a jet pump adds a considerable amount of fresh water, that means water without sediment, to the flow right at the beginning, you're absolutely sure to always have that much water available in the mixture. With proper choice of this drive discharge, the ladder can be let down into the sand, completely submerge itself, and you still do not run the risk of clogging your lines. In a 1-year operation of the prototype, it was found that as an average at least a 50 percent increase of the average concentration was possible compared to other dredges. Thank you.

MR. SIMMONS:

I think I can make mine very brief for two reasons. First, the majority of you will be in Vicksburg tomorrow, I understand, to see some of the work we have in progress, and the specific estuary model which will be on display will be the Savannah Harbor model, which was described by Mr. Harris this morning in his discussion. In addition to Savannah Harbor, we have a comprehensive model of New York Harbor, which was mentioned in the résumé of Mr. Panuzio's paper, and which has been used extensively to study plans designed to reduce sedimentation of the Hudson River navigation channels and the pier slips which abut this channel. I might emphasize the great importance of that pier slip shoaling problem, not only in New York Harbor, but in many other harbors. The maintenance of these facilities are not the responsibility of the Federal Government but the responsibility of the owners and operators of the facilities themselves. Dredging is quite an expensive proposition in these areas, because in most cases it has to be accomplished by clamshell type equipment or similar devices, with the spoil loaded on barges and hauled many miles to licensed disposal areas. As a consequence, the unit cost of dredging is much greater than has been quoted in connection with the Corps navigation projects, and in some cases runs as high as $1.50 per cubic yard.

Another semipermanent model at our installation is the Delaware River model, which reproduces that estuary from the Capes, where Delaware Bay joins the Atlantic Ocean, to Trenton, N. J., which is the head of tide in the Delaware. This model has been in existence for a great number of years, and from all indications it will be there for quite a long time to come. Incidentally, this model contributed its small bit to the decision that the sump rehandling method for disposal of dredge spoil was in fact the proper technique to be employed in the Delaware.

We have recently completed construction and adjustment of a model that reproduces the lower 52 miles of the Columbia River and pertinent offshore areas, which were discussed to a certain extent in the paper by Mr. Lockett. We are just getting into the study phase of this model, and apparently the problems involved are equally serious between the entrance, which deals primarily with rehabilitation of existing structures designed to improve flow conditions and minimize shoaling, and the upriver area, for which there has recently been authorized an increase in depth of the navigation channel. So I can foresee, in the course of this study, chasing back and forth from the entrance to the upriver areas every couple of weeks, trying to satisfy the design engineers with the information they need to carry on their work in the field.

We have recently completed construction of a deep draft navigation channel through Matagora Bay, Tex., extending from the gulf to Point Comfort, Tex. This study was rather unique in that this channel did not go through the process of evolution of most of our estuary channels, in which it was gradually deepened a few feet at a time from a natural depth to a present great depth. The channel will be dredged from a natural depth of about 12 feet to full project depth of 36 feet. I won't take the

time to mention any of the others, since those discussed represent a good cross section of the work we have in progress.

Professor Johnson handed me one question which reads as follows: Does good harbor maintenance encourage building of ships with even greater drafts, thus causing a continued compounding of the dredging problem in harbors? I think the answer to that is quite obvious, in that our experience with the commercial shipping interests indicate that they will build their ships to the maximum draft that navigation channels can accommodate, and this especially applies to the modern tankers. It is quite obvious that the unit price of transporting oil is lowest for the largest tankers, and that this factor is given major consideration in the design and construction of the modern supertankers. If navigation channels are maintained at 40 feet where the project says 40 feet rather than being allowed to shoal to lesser depth and then redredged to 40 feet, there is little doubt that vessels of greater draft will be developed to utilize these channels. Whether this is a good practice or not, I am not competent to say, but the economic benefits of that channel are thus improved if it can accommodate vessels of greater draft.

MR. CALDWELL:

I thought I would mention three things very briefly that are being done now or will be started in a short while. One is the study of inlet flow characteristics. This is part of the work of the tidal hydraulics committee of the Corps of Engineers which is sponsoring this work. We will try to define what might be called the discharge coefficient of inlet. Now we find that for navigation purposes and pollution control, the tendency is to want to open up the inlet. For the hurricane control, the tendency is to want to narrow or shut down on the inlet. Thus, we find that Fish and Wildlife Service usually would rather the inlets were not disturbed at all. So we find that we've got people working and pulling in all different directions, and we're not able to answer with definity many of the questions that arise about flow through inlets.

We're trying to do some work now to enable us to make more precise calculations of the effect of what you might call "monkeying around" with these inlets and what will happen back in the lagoons and bays fed by these inlets. Now the second piece of work I might mention ties in somewhat with Mr. Mauriello's very informative paper on dredges. We find that along the coast of the United States most of our important beaches are in an eroding condition. This is largely due to the tendency of waves to denude the beaches and much of this sand finds its way offshore or into the inlets where it's not wanted. Now we'd like to have this material back on the eroded beach, but we find that the pumping distances from the inlets where there may be a giant shoal are in many cases excessive. Right offshore in sight of these beaches there are, in some cases, tremendous quantities of sand, but so far there has not been a dredge developed that could operate out in the surf zone or just beyond the surf zone and pump this sand through the surf zone on to the beach.

One of our big projects right now is to try to develop a method of locating this sand, bringing in a suitable dredge, and pumping this sand through a pipe line through this surf zone on to the beach. Once we can perfect this method, a lot of our troubles will be over, because we will be able to find sand then (we hope at an economical price) and put it on the beach where it's most needed. The third thing I might mention is our present studies of dune rebuilding. Now this is a kind of a counterpart of what Professor Johnson was speaking of on sand movement, and here we are attempting to find the best ways to reconstitute a demolished dune line, particularly on our low barrier islands, such as the Carolina coast. There we are installing various types of sand fences, studying the resulting rate of growth of the dunes and what is the best arrangement.

Now I might mention that we have found from our recent experiments that an ordinary snow fence will fill with sand in as short a time as three or four weeks but that you gain nothing beyond that unless you install a second fence to cause this dune to build higher. Now at this low one-fence height, the dune is usually very vulnerable to any big storm that comes along. Thus, if we build it to one sand fence level, we know we should immediately go in and start building up fences to a greater height so that we can preserve and enlarge the dune. There will be periodic reports coming out, probably as technical memorandums of the Beach Erosion Board, on this last subject. Thank you very much.

MR. BRUUN:

Gentlemen, we have listened to a great number of papers informing us about the results of the latest research on sediment transport. Almost all of them dealt with river technology and represented the engineering approach emphasizing the solution of the most pressing practical problems. Progress has been made in the development of methods and in general understanding of river mechanism. Laboratory and field studies have brought to light the effect of sediment on the hydraulic characteristics of a stream and revealed the pertinent factors involved in stream geometry, the different

modes of transportation, and the great importance of bottom configuration and friction for the magnitude of sediment transport.

It is generally agreed that there is a strong need for research on bed forms and wash load transport. Special attention must be paid to the sediment problem at reservoirs and to the development of riverbeds, up- and down-stream of reservoirs and dams. The fact that good apparatus and techniques are readily available for sedimentation measurements makes it likely that results from various regions are generally comparable, and the laboratory engineer and researcher can point with a certain pride to the situation that field studies are applying laboratory results. We are, apparently, on the right track — as long as we are dealing with unidirectional river flow.

How is the situation on our seashores? Not nearly as good! Here we are faced with huge problems represented, e.g., by several million cubic yards of material to be dredged annually in order to maintain navigation entrances as the New York Harbor basins, the Columbia River, and the San Francisco Bay. On the beach erosion side of the problem: Florida is losing approximately 20 million cubic yards of sand per year, the Outer Banks in North Carolina are wearing out, and the dunes of Long Island and Cape Cod suffered severe damage during 1961–63 storms and earlier.

Progress in littoral drift technology has been hampered for several reasons. The two most important have been:

(1) Overwhelming problems.

(2) Unfortunately, the "experienced man." Meanwhile development of coastal engineering is still progressing, also in the fields of dredging and filling and in the erosion and protection technology. What we were not able to do a few years ago is now possible. "Zulia" dredged in 1 year 57 million cubic yards of fill by boom-operations from the Maracaibo Bar. Equipment for offshore dredging for beach nourishment is on the testing stage, and we may see it in actual operation before the end of 1963.

The "experienced man," who proceeded on a trial and error basis only, making one sand trap after the other, is now retiring slowly. It has been unlikely that any inlet or harbor would be established on the Continent without adequate model experiments. For years, very few studies with models have been accomplished in this country, but the San Francisco Bay studies and the Columbia River study are now proceeding. The Savannah River and Port problems have already been subjected to intensive studies. In Florida, no model study of navigation and beach erosion problems at our 17 most important inlets and passes had been carried out until 1956. In the spring of 1963 five studies had been completed, and three are in progress. So we are making headway there, too. New physical model laws for sediment transport, as proposed by Valembois, Yarlin, and as presented by Battjes and myself at this meeting, are expected to make model calibration and tests easier and more reliable. Meanwhile, we do not yet share the benefit of the tremendous achievements in adequate instrumentation for sediment transport technology by the river engineers, although progress has been made in wave and current recording methods. The efforts made by the U.S. Council on Wave Research in coordination and information services must be highly commended, but still, progress in littoral drift science and technology has been slow. In recent years, it seems that the tracing technique developed in England, Holland, and Portugal (radioactive tracing) and in the U.S.S.R. (fluorescent tracing) is going to mean a breakthrough in this important field. So far, tracing research has been of mainly qualitative nature, but quantitative results may be incoming in the near future.

This panel discussion is supposed to air developments and new ideas. It seems, therefore, to be proper to mention briefly the new scanning instrument for fluorescent tracing, which is being built at present for the Coastal Engineering Laboratory of Florida, financed by a grant from the Department of Health, Education, and Welfare.

The main problem associated with quantitative fluorescent sand tracing is the large number of samples that have to be analyzed and recorded in an extremely short period of time. Field and statistical experiments furthermore confirm that concentration of tracers within the samples will range between 10^{-2} and 10^{-7}. At either extreme, grave difficulties would be encountered if one would have to rely on manual and visual detecting, counting, and recording techniques alone, leaving too much room for repeatedly variable errors of some magnitude.

To comply with the test conditions producing samples at the approximate rate of 500 grams/minute, certain basic ideas have been developed at the Coastal Engineering Laboratory for the requirements and general plan of an automatic scanner for dealing with fluorescent tracers. On the basis of these ideas, a proposal was submitted by an electronic company for building such an instrument. In essence, the scanner as designed will complete a size analysis of samples of beach sand and differentiate within each sample several colors at the same time. The rate of scanning will occur at 20,000 particles per second.

The instrument essentially will consist of:

(1) A mechanical assembly of two parts —

a hopper for sand samples, and a Vycor glass tube in which the sand falls from the hopper at a variable controllable rate utilizing free-fall.

(2) An assembly of electronic circuitry. The sample falling in the tube passes in front of several optical channels enclosing photomultiplier cells (each cell in operation accounting for one of the many given colors in the sample), whereupon the photocell responds whenever a grain with the proper mercury-light-excited color crosses the light path of the optical channel. With a method of fixed-duration pulse-signal transmitting, a counter registers the signal (hence the colored grain) digitally or graphically after the signal passed through an emplifier, a pulse shaper, and a relay driver. Each photocell excludes all but one color as set by a combination of primary and secondary interference filters incorporated into the optical channels. The dimension of the output signal transmitted by the cell to the pulse shaper designates the size of each coated sand particle simultaneously with the fixation of its color. Within the counter, the tabulation of color, size, and quantity occurs at the rapid rate of 1.8 kg./hr.

The theory of the scanner is basically set in the following equation. The power entering the photocell is

$$P = I\sigma abQ \frac{1}{4\eta R^2} Acd \cdot H_\nu$$

where

I = Intensity of incident beam in photons
σ = Area of sand grain
a = Fraction of grain covered by fluorescent molecules
b = Fraction of irradiated fluorescent molecules which absorb incident photons
Q = Quantum efficiency of fluorescent process
R = Distance from sand grain to lens of optical system
A = Area of lens
c = Filter factor
d = Transmission factor for optical system
H_ν = Energy of an emitted photon.

Some limitations are inherent in this system. One is that the number of colors to be scanned simultaneously is dependent upon the spacing of their respective transmission curves within the visible spectrum with a minimum of overlap. At present four colors are contemplated for use while provisions will be made that the instrument could handle an additional four colors at a future date.

Since detection is proposed at peak outputs of each fluorescent curve through the use of narrow range filter, the question of how many colors could be employed remains an optico-mechanical task.

This instrument is one of the small steps forward, which will, we hope, help us to wind up with the excellent progress that has been made by our friend, the river engineer.

MR. JOHNSON:

Before turning the chair back to the chairman I will attempt to answer two questions which relate to my paper on sand transport by wind action. The first question was submitted by Professor Vanoni. It is, "Was there a windrowing affect on the sand; that is, was the mean size of the transported material less than that or those of the materials fed into the upstream and some among the bed?" The answer in general is, "yes." When the vertical trap which was one-half inch wide and extended a foot above the bed was exposed to low wind velocities and before the duration of the run was long enough for a scouring effect to occur around the trap, a comparison between the sand that was collected in the trap and that which was fed into the upper end of the channel were identical. At the higher windspeeds, some finer material not only went over the vertical trap but also passed over the horizontal trap and on through the fan. In general the agreement between the material trapped and that in the bed was good, and I would refer you to the work by Belly for the comparison. Professor Brooks asked the question: "Did any ripples or dunes appear in the wind tunnel and how did they affect transport?" The bed was always rippled during the tests. Runs were always long enough that an equilibrium condition occurred. The bed had very regular ripples, and we would not have had sand transport unless we had the ripples. A second question by Professor Brooks is: "In the field, how would one determine the shear velocity for a dune field which can be used in the transport equation?" In a limited number of field tests on dunes near Dillon Beach, Calif., we used a multiple pitot tube arrangement and measured the pressure difference on each tube by means of a Magnehelic gage. This instrument permits pressure differences of down to 0.01 inch of water to be measured.

Symposium 4.—Sedimentation in Reservoirs

INTRODUCTION

This symposium concerns problems directly related to sediment deposition in and above reservoirs. The papers and discussions to follow present typical problem situations encountered in the United States and describe practices used by government agencies having responsibilities for design and operation of reservoir projects. Individual papers discuss methods of estimating future conditions; surveying procedures; measuring instruments and techniques; design considerations; and measures used to alleviate certain problems. Most of the discussions pertain to problems associated with specific projects, specific types of reservoirs, or a single aspect of a reservoir sediment problem. All are concerned with one or more aspects of reservoir storage depletion and/or features associated with local deposition and erosion.

Information contained in the papers concerning reservoir surveys and related observations, reflect important improvements that have been made during the past 15 years in the development of instruments, equipment and techniques for measuring sediment. One of the most significant advances, implicit in several of the papers, has been the development of knowledge regarding optimum balance between requirements for survey facilities, survey types, and timing of resurveys necessary to accomplish the basic objectives.

Collectively, the papers show that the importance of problems varies regionally, depending upon the quantities and characteristics of materials transported by streams and according to reservoir types and purposes. However, reasonably accurate volumetric forecasts of total deposits or depletion of space within a given elevation-zone is not always indicative of the seriousness of problems that may arise. For example, storage allocated for recreation may become adversely affected for that purpose in a short period of time because of unfavorable distributions even in cases where total volumes of deposits are relatively small. It is noteworthy that several of the papers reflect the increased attention being given to sediment distributions in investigations for project planning, design and operation.

In view of the importance attached to locations of sediment deposits, it is significant that none of the papers contain suggestions regarding approaches for rigorous analyses to predict distributions. The distributions are governed by numerous factors which include sediment quantities and characteristics; reservoir size and shape; tributary locations and relative magnitudes of sediment contributions; unpredictable sequences of extremes of hydrologic events and effects of coincidental regulation operations; and influences of unforeseen upstream developments. Because of the complexities involved, the prospects for development of precise methods for general application in predicting future distributions are not promising. However, the papers show that, for engineering purposes, considerable progress has been made in the use of empirical methods to aid in foreseeing potential problems.

As the development of survey methods and techniques has progressed with field experience, improvement of capabilities for forecasting reservoir sedimentation effects, has been coincident with a gradual accumulation of improved basic data. The Subcommittee on Sedimentation has endeavored to facilitate continued efforts in reservoir sediment investigations by arranging for exchange of pertinent reports and records prepared by various agencies. A continuing activity in this area is a cooperative arrangement whereby standardized summaries are prepared of all reliable reservoir sedimentation surveys made in the United States. These records, which are now available for more than 1,000 reservoirs of various types and sizes, are expected to be of lasting value for further studies of problems represented in the following papers.

SEDIMENT STORAGE AND MEASUREMENT IN THE UPPER COLORADO RIVER BASIN

[Paper No. 78]

By CLYDE D. GESSEL, *chief, Special Studies Branch, Division of River Control, Bureau of Reclamation, Salt Lake City*

Abstract

Provision for sediment storage was a major consideration in the planning, selection, and design of the reservoirs of the Colorado River storage project in the Upper Colorado River Basin. These reservoirs are now all in final stages of planning, design, or construction, and include Flaming Gorge Reservoir on the Green River, Navajo Reservoir on the San Juan River, Blue Mesa, Morrow Point, and Crystal Reservoirs on the Gunnison River, and Lake Powell on the Colorado River. Total initial storage capacity is 33.6 million acre-feet. Estimated sediment accumulation will total 17.8 million acre-feet in 200 years. Extensive data collection facilities have been installed in the reservoir basins and in headwater and tailwater reaches to insure adequate means to accomplish periodic future measurements of deposited sediments and channel changes.

Introduction

Construction and closure of the four units of the Colorado River storage project will initiate a new stream regimen on the main stem of the Colorado River and on each of the prinicpal tributaries in the Upper Colorado River Basin. Gone forever will be the natural flow patterns. Largely substituted for the natural hydrographs of the uncontrolled streams will be flow patterns dictated by the requirements for storage, control, and use of the water of the Upper Colorado River to meet terms of the Colorado River Compact, the Upper Colorado River Basin Compact, and the various beneficial uses to which the waters of the basin will be applied. These include irrigation, industrial and municipal, power production, flood control, recreation, and fish and wildlife uses. With the control, storage, and use of the water will come attendant changes in the movement of sediment in the main stem of the Colorado River and principal tributaries in the Upper Colorado River Basin. This paper will describe some of the activities relating to the storage and future measurement of sediment in the four units of the Colorado River storage project. Time will not permit discussion of the numerous participating projects nor more than a general description of the problems involved and the planning that has been accomplished in handling the sediment phases of the project.

Provision for sediment storage was a major consideration in the planning, selection, and design of the principal reservoirs of the Colorado River storage project in the Upper Colorado River Basin. A basic concept adhered to was that a specified minimum active storage content should remain at the end of a 200-year period. Thus, reliable estimates were necessary to determine the storage capacity required in each of the project reservoirs to accommodate the amounts of sediment believed to be present and passing each of the damsites. Several estimates of sediment accumulation and distribution were made during various stages of planning the reservoir system as new data were collected and analyzed. These estimates have now been refined, based upon all data that are available. Further revisions will be made in the future as a result of continued measurement of sediment loads moving in the streams and observance of sediment accumulations in the reservoirs.

A brief description of each of the storage units will serve to acquaint you with the magnitude of the Colorado River storage project. Figure 1 shows the location of the reservoirs and principal geographic features of the Upper Colorado River Basin. Estimates of the amount of sediment that will be trapped in each reservoir will be presented, and the various means to measure the amount of sediment deposition and changes in stream channels in reservoir backwater and tailwater reaches will be described.

Description of Storage Units

Flaming Gorge Dam and Reservoir

Flaming Gorge Dam is located on the Green River in the northeastern corner of Utah about 40 miles north of Vernal. The dam and the lower part of the reservoir are situated in a deep gorge of the Uinta Mountains, a part of which was named Flaming Gorge by John Wesley Powell in his exploration of the Green and Colorado Rivers in 1869. The rest of the 90-mile-long reservoir extends upstream into the rolling hills and flat lands of southwestern Wyoming to the vicinity of Green River, Wyo. The reservoir will control a drainage area of 15,350 square miles and contain total storage of 3,789,000 acre-feet when full to normal water surface level of 6,040 feet m.s.l. Operation of Flaming Gorge will be geared for the maximum

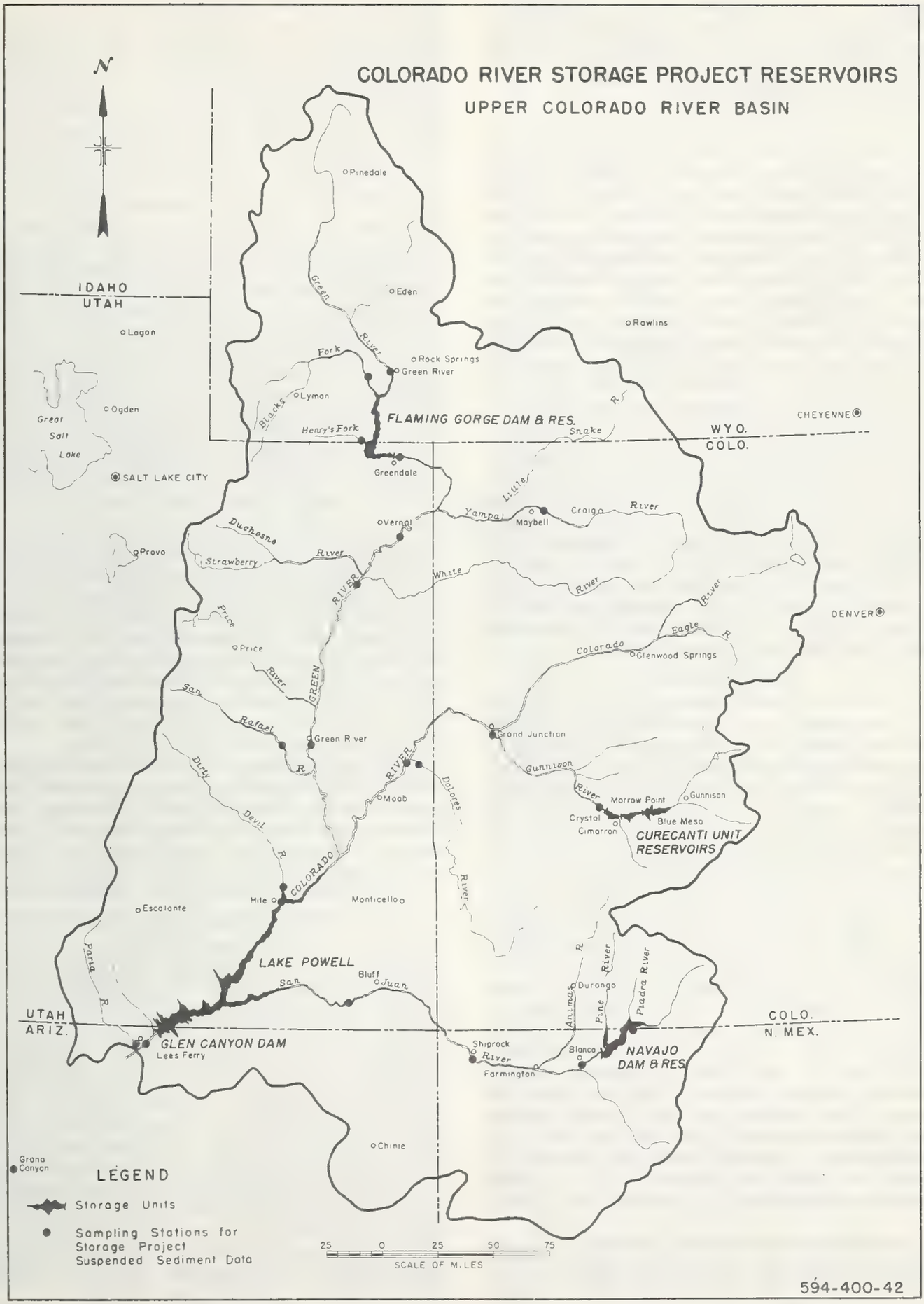


FIGURE 1. — Reservoirs and principal geographic features of the Upper Colorado River Basin.

generation of hydroelectric energy, so considerable seasonal and long term fluctuation of the water surface is expected. Flaming Gorge Dam was closed, and storage in the reservoir began on November 1, 1962.

Navajo Reservoir

Navajo Reservoir was the first storage project unit to be closed and storage initiated therein. This occurred on June 27, 1962. Navajo Dam is located on the San Juan River immediately downstream from its junction with the Pine River in northwestern New Mexico. The reservoir extends 35 miles upstream into Colorado, and the headwaters are situated in both the Piedra and San Juan Rivers at normal water surface elevation of 6,085 feet m.s.l. The drainage area is 3,190 square miles, and storage capacity totals 1,709,000 acre-feet. No powerplant was included, but the dam will form the future diversion point for the Navajo Indian irrigation project south of the San Juan River in the vicinity of Farmington, N. Mex. The bulk of the river flow will be released through the outlet works initially until major diversion demands arise for the irrigation project. This reservoir will largely regulate the San Juan River at the damsite, but sizable spills will occur in years of high runoff and several large tributaries of the San Juan River below Navajo Dam will not be regulated.

Curecanti Unit

The Curecanti unit consists at the present time of two reservoirs located on the Gunnison River downstream from Gunnison, Colo., and upstream from the mouth of the Cimarron River. Blue Mesa Reservoir, formed behind Blue Mesa Dam, is the upper and principal regulatory unit, with a storage capacity of 941,000 acre-feet at elevation 7,519.4 feet m.s.l. Morrow Point Dam, 12 miles downstream from Blue Mesa Dam, forms a 117,000-acre-foot lake in the deep, narrow Gunnison River Canyon. Crystal Dam and Reservoir, next downsteam, with a storage capacity of 38,000 acre-feet, is now under active consideration. All three dams will be equipped with hydroelectric powerplants. Peaking energy will be generated at Blue Mesa and Morrow Point powerplants. Crystal Reservoir will thus be required to provide short-term regulation of releases down the Gunnison River, while Blue Mesa Reservoir will possess sufficient capacity for seasonal and long term regulation. Completion of Blue Mesa Dam is due in late 1965, and Morrow Point in 1966. No definite schedule is yet available for Crystal Dam.

Glen Canyon Dam and Lake Powell

Lake Powell, the reservoir behind Glen Canyon Dam, is the largest reservoir of the storage project. It was appropriately named after John Wesley Powell, who, as previously mentioned, first explored the Colorado River in 1869. Glen Canyon Dam is located about 15 miles upstream from the boundary between the Upper and Lower Colorado River Basins, and the reservoir extends upriver 186 miles. In many ways, Glen Canyon Dam and Lake Powell closely resemble world famous Hoover Dam and Lake Mead. Glen Canyon Dam towers approximately 710 feet above bedrock as compared to Hoover Dam's 726 feet. Lake Powell will provide a total of 27 million acre-feet of storage at normal water surface elevation of 3,700 feet m.s.l., as compared to Lake Mead's initial capacity of 31,250,000 acre-feet.

Lake Powell is assigned the task of final regulation of the Colorado River before the stream leaves the Upper Basin. This long-term storage requirement, together with maximum power production, will cause the lake to experience cycles of filling and emptying extending over many years, similar to the pattern of operations at Lake Mead. Also, as in the case of Lake Mead, Lake Powell will be required to store a tremendous load of sediment annually. Headwaters of both reservoirs are buried in deep, narrow canyons. This is fortunate in some ways, as reservoir fluctuations will prevent formation of large exposed deltas and consequent phreatophyte problems. At the headwaters Lake Powell is divided between the main stem and its major lower tributary, the San Juan River, in some respects similar to the Colorado-Virgin River division of Lake Mead.

Construction began on Glen Canyon Dam in October 1956 and has now progressed to the point where closure of the diversion tunnels and storage of water in the reservoir are imminent. Completion is scheduled for 1964.

Sediment Storage in Reservoirs

Flaming Gorge Reservoir

Studies of suspended sediment in the Green River in the vicinity of Flaming Gorge Dam and Reservoir revealed that we could expect a mean annual accumulation of 2,040 acre-feet. These studies were initially based upon suspended sediment sampling of streams tributary to the reservoir. Sediment measuring points included the Green River at Green River, Wyo.; Blacks Fork near Green River, Wyo.; and Henrys Fork near Linwood, Utah. Further comparisons were made by utilizing downstream records on the Green River near Jensen, Ouray, and Green River, Utah, and the Yampa River at Maybell, Colo., to establish contribution rates for unmeasured areas. Final checks were completed after 2 years of suspended sediment sampling on the Green River near Greendale, Utah.

These comparative studies all confirmed the estimate of 2,040 acre-feet of annual sediment accumulation with the bulk of this amount originating in the Blacks Fork drainage and other tributaries between Green River, Wyo., and the damsite. Sediment load in the Green River at Green River, Wyo., proved to be comparatively light.

Navajo Reservoir

The San Juan River sediment load at the Navajo Dam was calculated to be 3,300 acre-feet annually. This figure is a net inflow to Navajo Reservoir aftereffects of the recently authorized San Juan-Chama transmountain diversion from the headwaters of the San Juan River were accounted for. Studies were based upon several years' record of the San Juan River at Blanco, N. Mex., and by comparison with other long term records on the river. The dam lies upstream from the heaviest sediment-producing areas in the San Juan River drainage.

Curecanti Unit

The reservoirs of this unit are located on the exceptionally clean, sedimentwise, upper Gunnison River. Estimates indicate that less than 340 acre-feet of sediment would accumulate annually in Blue Mesa Reservoir and about 28 acre-feet in Morrow Point. The proposed Crystal Reservoir will receive and store approximately 38 acre-feet annually, mostly from the Cimarron River drainage.

Lake Powell

Far and away the largest sediment load affecting the storage units will be carried by the Colorado and San Juan Rivers into and stored in Lake Powell, the principal regulatory reservoir of the Upper Colorado River Basin. Lake Powell is situated below the great sediment-producing areas in Colorado, Utah, New Mexico, and Arizona, tributary to the Upper Colorado River Basin. Fortunately, extensive suspended sediment records, some of which date back to 1925, are available for the Green, San Juan, and Colorado Rivers above the reservoir and at Lees Ferry and Grand Canyon below the dam. After careful consideration of the suspended sediment and water records and the possible effect of major authorized upstream storage units, we estimated that an average of 104,000 acre-feet of sediment could be expected to flow into Lake Powell annually. Compaction of the sediments in a 100-year period would reduce the 104,000 acre-feet to a mean annual volume of 85,400 acre-feet. Thus, the storage requirement for sediment was calculated to be 8,540,000 acre-feet for a 100-year period. Further compaction results in a 16,800,000-acre-foot requirement for a 200-year period.

Possible long term effects of participating project developments were not considered. This figure also takes full cognizance of and is partially dependent upon records of the main Colorado River back to 1925, although records since 1943 indicate considerably less sediment was carried annually in the Colorado River since 1943 than the mean annual load previous to that year. Consequently, we feel this amount is an adequate reflection of any likely long-term sediment-producing condition on the Colorado River. The actual measured accumulation in Lake Mead for the period 1935-48, inclusive, also offers a check on our calculations.

Special Problems and Plans for Future Measurement of Sedimentation

Flaming Gorge

Storage of the relatively small amount of annual sediment inflow offers no particular problems for Flaming Gorge Reservoir, as the 200-year sediment load of 398,000 acre-feet is expected to occupy only about 10 percent of the total reservoir capacity. A special topographic map of the reservoir basin has been completed that will provide basic reference data for any needed remeasurement in the reservoir basin. Bedload and delta formation in the headwaters of the reservoir on the Green River and on Blacks Fork were investigated to predict the possible backwater effects on improvements near the reservoir.

The city of Green River, Wyo., lies on the bank of the Green River, and the Union Pacific Railroad and Highway 30 cross the Green River near the city about 7 miles upstream from the normal water surface of Flaming Gorge Reservoir. An extensive network of 27 permanent ranges was installed beginning at the railroad bridge and extending downstream to where the last six ranges lie in the reservoir basin. These ranges were surveyed to provide data for backwater studies, with an expected sediment delta in place and for future measurement of delta formations and channel changes in the headwaters of the reservoir and in the channel reaches upstream to Green River. Studies showed no backwater effects from the reservoir would extend upriver to threaten any of the improvements previously mentioned.

A short section of Wyoming State Highway 530 and the highway bridge across Blacks Fork on the west side of Flaming Gorge Reservoir were required to be raised because of water backing up Blacks Fork from the reservoir. The present location offered some advantages for the crossing rather than crossing above the reservoir line. Blacks Fork carries about 750 acre-feet of sediment per year into Flaming

Gorge, and a considerable portion of this is expected to form a delta in the Blacks Fork channel where little storage space is available. Consequently, studies were completed to establish minimum clearance for the highway bridge under conditions of delta formation and expected flood flows. Fifteen sediment ranges were surveyed to furnish required data. All ranges were permanently monumented.

Navajo Reservoir

Construction of the Navajo Dam and Reservoir required the relocation of the Denver & Rio Grande Western's narrow gage railroad and State and county roads near the head of the reservoir along the San Juan and across the Piedra Rivers. Sediment delta and backwater studies were completed after additional suspended sediment records were gathered and permanent ranges installed. The data were then utilized to select locations and elevations of roadbeds, including the bridge crossings of the Piedra River.

Expected deposition of sediment in Navajo Reservoir will total about 532,000 acre-feet in a 200-year period and will occupy about 31 percent of the initial capacity. This estimate indicated the need for accurate periodic remeasurement of the reservoir basin to adjust area-capacity data and operation schedules.

To best fill this need, a topographic map with 10-foot contours was obtained and a comprehensive network of permanent sediment ranges was designed and installed throughout the reservoir basin and headwater delta reaches. This network consists of 82 permanently monumented ranges at about 1-mile intervals linked together by a common survey and based upon national systems of horizontal and vertical control. These ranges will provide adequate basic data for reservoir resurveys at any time required by project operations.

Regulation of the San Juan River and relief of its sediment load in Navajo Reservoir may cause changes in the channel below the dam. Many side canyons and washes draining to the San Juan are subjected to flash floods and occasionally transport and deposit of heavy loads of sediment in the San Juan River channel. Thus, the San Juan River channel could experience degradation in some sections of the channel and aggradation in other parts of the reach from Navajo Dam to Shiprock, N. Mex. To measure these channel changes, a system of river ranges is now being installed downstream to Shiprock, with the bulk of the ranges occurring between the dam and Blanco. Many of these ranges will be below the mouths of tributary canyons, where changes in channel conditions seem most likely. Additional cross section data are available at the several Geological Survey stations along the river, and the State of New Mexico has expressed interest in surveying additional ranges or obtaining new aerial photographic coverage of the river channel.

Curecanti Unit

Authorizing legislation for the Colorado River storage project specified that a storage reservoir of not less than 940,000 acre-feet should be constructed on the Gunnison River, or such greater capacity as could be obtained with a water surface at 7,520 feet m.s.l. at the Blue Mesa site. A water surface elevation of 7,519.4 feet m.s.l. was needed to provide 941,000 acre-feet of storage. Prospective delta formation was believed to be minor for Blue Mesa Reservoir, but backwater studies with delta formations were made to determine if any encroachment would take place. Calculations showed that reservoir and headwater channel clearing of brush and trees would offset any rise in water surface elevations caused by this minor delta formation. Sediment ranges were established by permanent markers.

The Cimarron River enters the Gunnison in the reach common to the tailwater of Morrow Point and headwater of the prospective Crystal Reservoir. Studies have now been completed on delta formation in this reach to furnish needed design data.

Expected reservoir sediment depositions will not cause significant deleterious effect in Blue Mesa or Morrow Point Reservoirs, but sediment will occupy upward of 20 percent of the total storage in Crystal Reservoir in a 200-year period.

Lake Powell

The accumulation of sediment expected in Lake Powell, as described previously, constitutes a noteworthy factor in the operation and management of the reservoir. The 200-year sediment accumulation of 16,800,000 acre-feet will eventually occupy about 62 percent of the initial capacity. Therefore, careful consideration was given to provide adequate basic data before the reservoir filled that would permit periodic measurement of the accumulated sediments and also be used for research purposes as the need arises for additional studies. The rugged and inaccessible walls of Glen Canyon of the Colorado River soon convinced us that the conventional method of field surveying and installation of sediment ranges in Lake Powell would not be practical or economical. Consideration was then given to establishment of sufficient basic vertical and horizontal control that would permit construction of sediment ranges or strips of topography across the reservoir by photogrammetric means. An alternate to this latter scheme was a proposal to remap the en-

tire reservoir basin at a large scale and small contour interval.

The remapping plan was finally adopted, as its estimated cost was approximately the same as for providing sediment ranges photogrammetrically. The complete map offered much more freedom for remeasurement for sediment purposes and also furnished a large-scale map for an accurate initial determination of area and capacity data and for many other uses associated with the development and management of the reservoir. I have indicated the topographic coverage was a "remapping" procedure. Modern map coverage was available for Lake Powell but only at a scale of approximately 1 inch equals 1 mile and with a contour interval of 80 feet. Specifications for the new map called for a scale of 1 inch equals 400 feet and contour interval of 10 feet.

The cooperative efforts of three agencies of the Federal Government and several private firms produced a map of high standard that covered the entire reservoir to 3,750 feet m.s.l., some 50 feet above the normal water surface. The excellent vertical photographic coverage of the natural reservoir basin is also a fine byproduct. This mapping required unique applications of vertical control procedures and called for large scale use of helicopter transportation to establish control surveys. The complete mapping job is described in the article by Gessel and Rutledge.[1]

Extensive first-order level lines joined to the national leveling network were also established throughout the reservoir basin, to make available precise data for measurement of sediment deposition in the future and to allow installation of ranges when operating personnel will be able to work from the lake surface. These high-order level lines will also allow detection and measurement of possible displacement of the earth's crust in the reservoir basin due to the concentration of approximately 37 billion tons of water and sediment.

Predictions of the behavior of the Colorado River channel below Glen Canyon Dam were needed to determine design data for the dam and powerplant. It appeared that the Paria riffle, some 15 miles downstream, offered the closest stable downstream control section, so 21 ranges were surveyed in the reach between the dam and the mouth of the Paria River. These ranges have been resurveyed several times and revealed some heavy degradation in the stream channel. Other resurveys will be made as required.

[1] GESSEL, C. D., and RUTLEDGE, D. H. LARGE SCALE MAPPING OF LAKE POWELL. Amer. Soc. Civil Engin., Jour. of the Surveying and Mapping Div., Paper 3318, 17 pp. 1962.

Long range planning for recreational development on Lake Powell required consideration of sediment aspects. The National Park Service was interested in establishing a major recreational development near the present Hite Ferry crossing in the upper reaches of Lake Powell. After consultation and study by the Bureau of Reclamation, it was shown that an installation at this location would have a comparatively short life due to sediment delta encroachment. The Bureau study made for this purpose drew heavily from data and experience of Lake Mead, where the sediment delta has now progressed downstream and largely engulfed the Pierce basin..

A resurvey of the delta in Lake Mead in early 1961 revealed a topset slope of 1 foot per mile. This topset slope data, together with the predicted amount of sediment inflow from the Colorado River arm and the cross sectional data available from the new topographic map resulted in an estimate of 20 to 25 years before the Hite basin would be engulfed and rendered hazardous for recreational purposes.

Another location accessible by road near the mouth of Red Creek about 12 miles downstream would likely survive for a 40- to 50-year period. We understand the Park Service is considering this site. Use of an electronic data computer greatly speeded the several trial studies required, as additional studies were computed rapidly after initial data preparation was complete. Guidance was also given to solve a similar problem on the San Juan arm of the reservoir. Here again, it was revealed that the most readily accessible area on the San Juan part of Lake Powell will be affected by sediment shortly after storage in the reservoir begins.

Closure of the coffer dam and diversion of the Colorado River in February 1959 has caused significant amounts of sediment to deposit in Lake Powell during construction of Glen Canyon Dam. This is evidenced by the greatly reduced measurements of suspended sediment in the Colorado River at Lees Ferry and the visible deposits in the reservoir basin. Quantitative measurements were not taken in the reservoir basin, but, from study of flow and sediment records at Lees Ferry, we estimate that deposits in the order of 60,000 acre-feet accrued in the 1959–61 water years. Comparison of records on the tributary streams above the reservoir basin with records for Lees Ferry in 1960 also showed deposition occurred. The total suspended load for 1960 at the three principal stations of San Juan River near Bluff, Colorado River near Cisco, and Green River at Green River, Utah, was 116 percent of the load at Lees Ferry. The highest percentage for

previous years was 79 in 1954, while the average was 62 percent for the 1943-58 period.

Discussion

To summarize, it is apparent that in a space of less than a decade the stream regimen in much of the Upper Colorado River will be significantly changed as the reservoirs of the Colorado River storage project will regulate the principal tributaries and store great quantities of sediment. Lake Powell, by far the largest and most important reservoir, will bear the greatest burden. It will trap 16,800,000 acre-feet of sediment in 200 years. Navajo Reservoir will accumulate 532,000 acre-feet in the same period, while the sediment loads in Flaming Gorge and the Curecanti unit reservoirs are relatively minor when judged against the capacities and functions of these reservoirs. Overall, the storage project reservoirs will be called upon to store about 17.8 million acre-feet of sediment in 200 years, or 53 percent of their total initial capacity (table 1). Many studies have been completed to solve special problems associated with sediment. Data collection facilities have been installed that will provide information for future operational and research purposes.

We intend to watch the sediment phases of this project with great interest in the coming years and accomplish necessary data gathering and additional studies as the needs arise.

TABLE 1.—*Estimated sediment accumulation, Colorado River storage project reservoirs, 1963*

Reservoir	River	Total capacity	Annual sediment inflow	100-year sediment accumulation[1]	200-year sediment accumulation[1]	200-year condition, percentage of initial capacity[1]
		Acre-feet	*Acre-feet*	*Acre-feet*	*Acre-feet*	
Flaming Gorge	Green	3,789,000	2,040	201,000	398,000	10.5
Navajo	San Juan	1,709,000	3,300	276,000	532,000	31.1
Blue Mesa	Gunnison	941,000	340	33,300	66,300	7.0
Morrow Point	do	117,000	28	2,700	5,400	4.6
Crystal	do	38,000	38	3,800	7,500	19.7
Lake Powell	Colorado	27,000,000	104,000	8,540,000	16,800,000	62.2
Totals		33,594,000		9,056,800	17,809,200	53.0

[1] 100-year and 200-year figures are compacted sediment amounts.

SEDIMENTATION IN WALLA WALLA ARM OF McNARY RESERVOIR AND ITS EFFECT ON RECREATIONAL DEVELOPMENT

[Paper No. 79]

By MELVIN J. ORD, *chief, Water Control Section,* and JAMES C. CANNON, *head, Hydrography and Sedimentation Subsection, U.S. Army Engineer District, Walla Walla*

Synopsis

The preliminary master plan for use of the reservoir created by McNary Dam on the Columbia River provided for a recreational park to be developed by the State of Washington where the Walla Walla River enters the reservoir. Subsequent investigations led to the abandonment of the site as a State park because of predicted deposition of sediment carried by the Walla Walla River. Sediment ranges were established and five surveys have been made since impoundment of water in 1953. Results of these surveys through 1961 show an average deposition rate of 860 acre-feet per year. This indicates the water storage area below normal pool, elevation 340 feet m.s.l., in the Walla Walla arm will be essentially filled with sediment by the fall of 1965, except for the river channel. The deposition rate is equivalent to 0.49 acre-foot per year per square mile of drainage area. Plans have been investigated to protect portions of the water area for boating and swimming by construction of dikes and intermittent dredging, but as yet these have not proved feasible. As deposition progresses it will enter into the main reservoir and may adversely affect navigation in the area being developed by Walla Walla Port district. Sedimentation also is an important factor in planning other recreational developments along the Columbia and Snake Rivers.

Introduction

This paper presents the results of a sediment study of the Walla Walla arm of McNary Reservoir and the effect of this sedimentation on recreational development.

The initial plans for recreational development of the reservoir (Wallula Lake) created

by McNary Dam provided for a recreational park to be developed in the area where Walla Walla River enters the reservoir. The Walla Walla River drains a large agricultural area, much of which is devoted to production of wheat and peas. Heavy rains or rapid snowmelt and thawing of frozen ground in spring cause large quantities of sediment to enter streams tributary to Walla Walla River, and much of this sediment is carried down to the mouth of the river.

Based on the limited data available before impoundment, it was predicted that most of the water area in the Walla Walla arm would fill with sediment in a relatively short time. Plans for the State park development therefore were abandoned. However, it was desired that provisions be made for as much recreational use of the area as practicable. Sediment ranges were established and have been periodically surveyed since impoundment of water in 1953 to determine the distribution and rate of deposition. The results of surveys made to date are presented, including preliminary information on the relationship of sediment accumulation to streamflow and drainage area. Also, general information is presented on the influence of sedimentation in the planning of recreational developments at some other locations in the Walla Walla District of the Corps of Engineers.

Description of Area

McNary Dam and Reservoir

McNary Dam is located 292 miles above the mouth of the Columbia River, 161 miles east of Portland, Oreg., and 2½ miles upstream from Umatilla, Oreg. (fig. 1). About one-half of the dam is located in Washington and one-half in Oregon. The dam is constructed of concrete and earth and was completed in 1953. Overall length of the dam is 7,365 feet. Height above streambed is 100 feet, and the normal pool elevation of the dam is 340 feet above mean sea level. Backwater effects extend about 62 miles upstream on the Columbia River, 10 miles upstream on the Snake River, 9 miles up the Walla Walla River, and 6 miles up the Yakima River. The greatest width of the reservoir is about 2 miles and is near the mouth of the Walla Walla River which enters the reservoir on the left side about 25 miles upstream from McNary Dam.

McNary Lock and Dam is one multiple-purpose unit of the main control plan for the comprehensive development of the Columbia River and tributaries. Its primary uses are power and navigation with secondary benefits from flood control, irrigation, recreation, industrial site development, and other miscellaneous benefits. Pondage of the reservoir may be used on a regular schedule for peak power production, with drawdowns of 2 or 3 feet in the ordinary operation of the project and a maximum drawdown of 5 feet. Since impoundment of water in 1953 the maximum fluctuation in water surface elevation at the mouth of Walla Walla River has been about 5 feet. Maximum fluctuations of reservoir elevations at the mouth of the Walla Walla River are shown in figure 2.

Walla Walla River

Basin description

The Walla Walla River rises in the rugged Blue Mountains near Walla Walla, Wash., and flows about 75 miles westerly before emptying into the reservoir (fig. 3). The Touchet River is the principal tributary and rises in another part of the Blue Mountains near Dayton, Wash. It flows southwesterly for 75 miles before joining the Walla Walla River near Touchet, Wash. The Walla Walla River drains an area of about 1,700 square miles located in northeastern Oregon and southeastern Washington. The watershed from the reservoir to the mountains is predominantly an agricultural area. The hill and valley plain lands are extensively utilized for agricultural purposes, primarily for the production of wheat, barley, and peas. Land used for grain and peas is the primary source of sediment. The practice of cultivating much of this grainland in the fall increases the susceptibility to erosion. The higher slopes of the Blue Mountains are timbered and produce little sediment.

The climate is characterized by moderate temperatures and a large annual range in temperatures, with light to moderate precipitation. Average normal temperatures at Walla Walla range from 33° F. in January to 74° in July. Extremes have ranged from minus 29° to 110°. Mean annual precipitation ranges from about 8 inches near the mouth of the Walla Walla River to about 45 inches in the Blue Mountains, and averages 21 inches over the basin. Summer precipitation is usually very light. High rainfall intensities are infrequent; the highest observed was 2.4 inches in 40 minutes.

The Walla Walla River basin is underlain almost entirely by Columbia River basalt flows, which have been somewhat folded and locally faulted. Occasional sedimentary interbeds are found, and the basalt is overlain locally by gravels and generally by loess, with a minimum thickness of 10 to 15 feet. Alluvial fans are present near the mountains and are comprised of relatively homogeneous sediments consisting principally of subrounded pebbles and cobbles often interbedded with sand containing some silt. These deposits are pervious and become partially "silted in," but there is always considerable downward percolation to the water

FIGURE 1. — Columbia River, Oregon and Washington.

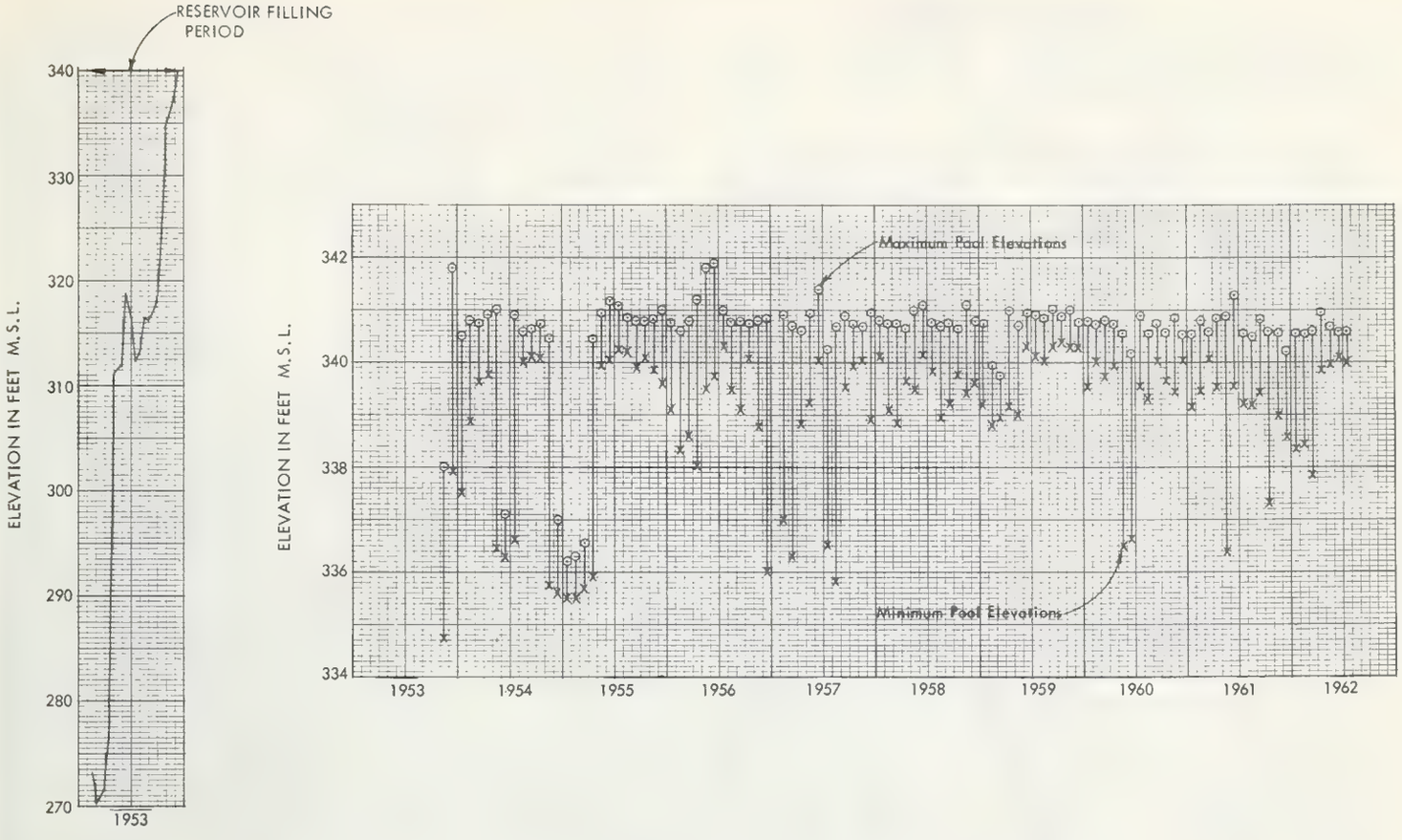


FIGURE 2. — Monthly reservoir elevations, mouth of Walla Walla River.

table, which underlies the surface at a depth of about 30 feet. Stream aggradation is continuing throughout the fan areas.

Streamflow

In general, the streamflow characteristics of the Walla Walla River basin are low flows from July through November, with occasional high flows in December, January, and February. On the average, the highest annual flows occur during the period March through June. Major floods in the area result from rainfall or a combination of rainfall and snowmelt on saturated or frozen ground conditions. Rapid thaw of the top layer of frozen ground during these occurrences results in heavy erosion of cultivated land and high concentration of sediment in the Walla Walla River. When snowmelt augments rainfall runoff, the floods have both high peaks and large volumes.

Walla Walla River below Touchet River produces an estimated average annual runoff of 440,000 acre-feet, or 5 inches over the 1,660-square-mile drainage area, based on brief records at this location and longer records in headwater areas and other streams. Individual yearly runoffs vary with annual precipitation received, and based on above records very likely have ranged from about 60 to 180 percent of the mean. Streamflows vary greatly during the year. The long-term mean flow is about 600 c.f.s., but in the brief period of record at the gaging station below Touchet River flows of 16,300 c.f.s. and 11 c.f.s. have been recorded. Flows have a seasonal pattern, with consistently low flow during the summer and fall months and during cold weather in the winter months. Table 1 is a summary of discharge data for the period since installation of stream gage in 1953.

TABLE 1.—*Summary of runoff data for Walla Walla River near Touchet, Wash., 1953–61*

[Drainage area, 1,600 square miles]

Month	Mean	Monthly runoff		Daily runoff	
		Max.	Min.	Max.	Min.
	C.f.s.	*C.f.s.*	*C.f.s.*	*C.f.s.*	*C.f.s.*
January	975	1,888	334	6,040	200
February	1,318	2,096	593	4,860	250
March	1,211	1,843	518	3,730	282
April	1,290	2,165	932	5,610	412
May	852	1,217	298	2,770	186
June	241	471	142	1,680	8.8
July	27	46	7	88	2.3
August	15	34	4	78	2.1
September	50	181	14	850	5.8
October	126	336	63	540	37
November	305	531	124	1,510	72
December	785	1,807	287	5,950	180

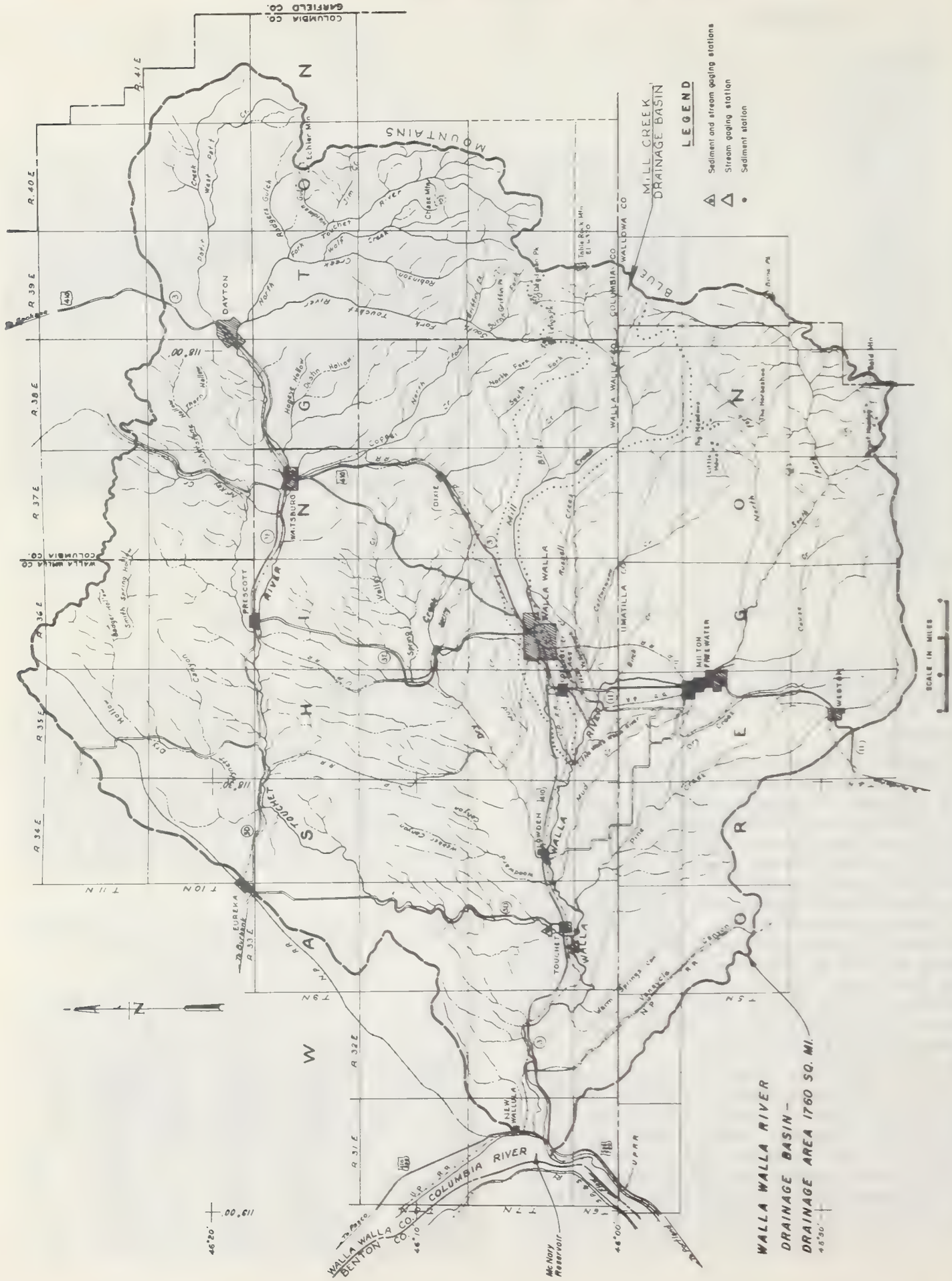


FIGURE 3. — Walla Walla River drainage basin.

Moderately high flows occur for several weeks in the late winter and spring as a result of snowmelt, and the larger part of the normal year's runoff volume is from this latter source. The stream slopes vary from 100 feet per mile in the upper reaches of the stream to 35 feet per mile in the lower reaches.

Sediment Sampling

Suspended sediment samples were obtained at two stations in the Walla Walla River basin on a regular basis during the period 1951 through 1953. These stations were located respectively on the Walla Walla River and the Touchet River immediately above their confluence (fig. 3). The suspended samples were obtained with a D–43 sampler. Concentration of sediment varied from 0 to 45,000 p.p.m. on Walla Walla River and from 0 to 82,000 p.p.m. on Touchet River. Graphs of the flow of the streams and concentration of sediment are shown in figures 4 and 5.

The somewhat erratic nature of the relation between discharge and sediment content is indicated by comparison of the two graphs. This characteristic is explained by the variation in soil and runoff conditions that exist during the year. Also, it will be noted from figures 4 and 5 that most of the heavy concentration of sediment occurs during relatively short periods during the winter and spring. These represent periods of heavy runoff from the agricultural area caused by rains, rapid snowmelt, rapid thaw of frozen ground, or a combination of these factors. The gradual snowmelt runoff from the higher elevation area does not carry much sediment.

No measurement was made of the movement of bedload other than could be measured by D-43 sampler. However, observations at the bed at low water and analyses of suspended, bed, and deposited material (figs. 6, 7, 8) indicated that the bedload was relatively small, and therefore it was estimated at 10 percent of the suspended load. The total average annual sediment load during this 3-year period was estimated at 1,540 acre-feet, as shown in table 2.

The average annual runoff for this 3-year period is only slightly below the long-term average. Consequently, the average annual sediment load of 1,540 acre-feet may be considered fairly representative of that for a long-term period.

TABLE 2.—*Annual runoff and sediment load, Walla Walla River, 1951–53*

Year	Runoff	Sediment load[1]
	Acre-feet	*Acre-feet*
1951	360,000	1,310
1952	419,200	1,880
1953	462,100	1,420
Average	413,800	1,540

[1]From sediment samples; includes 10 percent bedload.

Analyses were made of selected samples of the suspended and bed materials at the above stations and at accessible locations on the sediment delta between ranges 4 and 5. Typical gradation curves are shown in figures 6, 7 and 8. Average of gradation curve values show about 80 percent of the sediment to be silt and clay.

Sediment Surveys

As a basis for measurement of the rate and distribution of sediment deposits, 15 sediment ranges were established during 1951 prior to impoundment of water in the reservoir. During 1958, six additional ranges were established downstream of range 1 to extend the measurements out into the main body of the reservoir. The locations of 9 of the 15 original ranges are shown in figures 9 and 10. All ranges were marked with concrete monuments to establish a permanent system for periodic surveys.

Five sediment surveys have been made since establishment of the 15 ranges. Of these, 3 surveys were completed for the 15 original ranges and the 6 ranges established in 1958. The other two surveys have been confined to the ranges in the area receiving major deposits. Soundings were made with a fathometer. Targets and transit were used for keeping the moving boat on line. The ranges and cut-in points located to give suitable cut-in angles are plotted on plane table sheets. Check points along the continuous range profiles are made by fixes on fathometer charts that correspond with alidade intersections of the range line on plane table sheet coordinated by portable radio. Figures 11 and 12 show survey crew and equipment. The 4-wheel drive vehicle, boat trailer, 16-foot jet-propelled sounding boat, and crew are shown in the process of launching in figure 11. A close-up view of the fathometer is also shown. Figure 12 shows plane table being oriented over cut-in point and boat proceeding to range location to start sounding operations.

Sediment Deposits

Analyses of sediment range surveys show rapid filling of the water area in the Walla Walla arm of the reservoir. As would be expected, heaviest deposits started at the head end of the reservoir arm and have progressed each year toward the main reservoir. The progressive deposition at each range is shown in figures 13, 14, and 15. No appreciable deposition has occurred downstream from range 1 or upstream from range 9. Depths of deposit

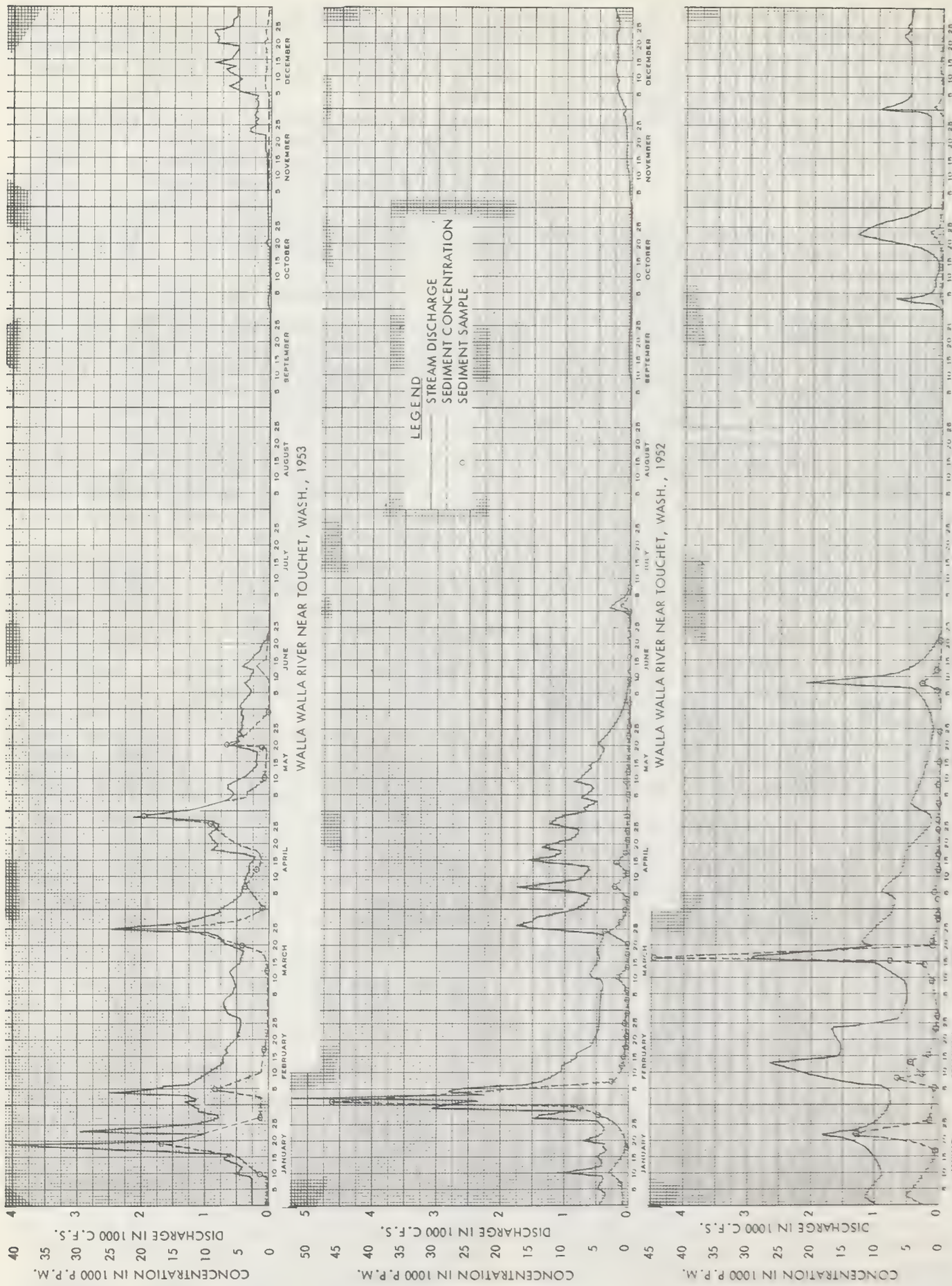


FIGURE 4. — Relationship of sediment concentration to stream discharge, Walla Walla River.

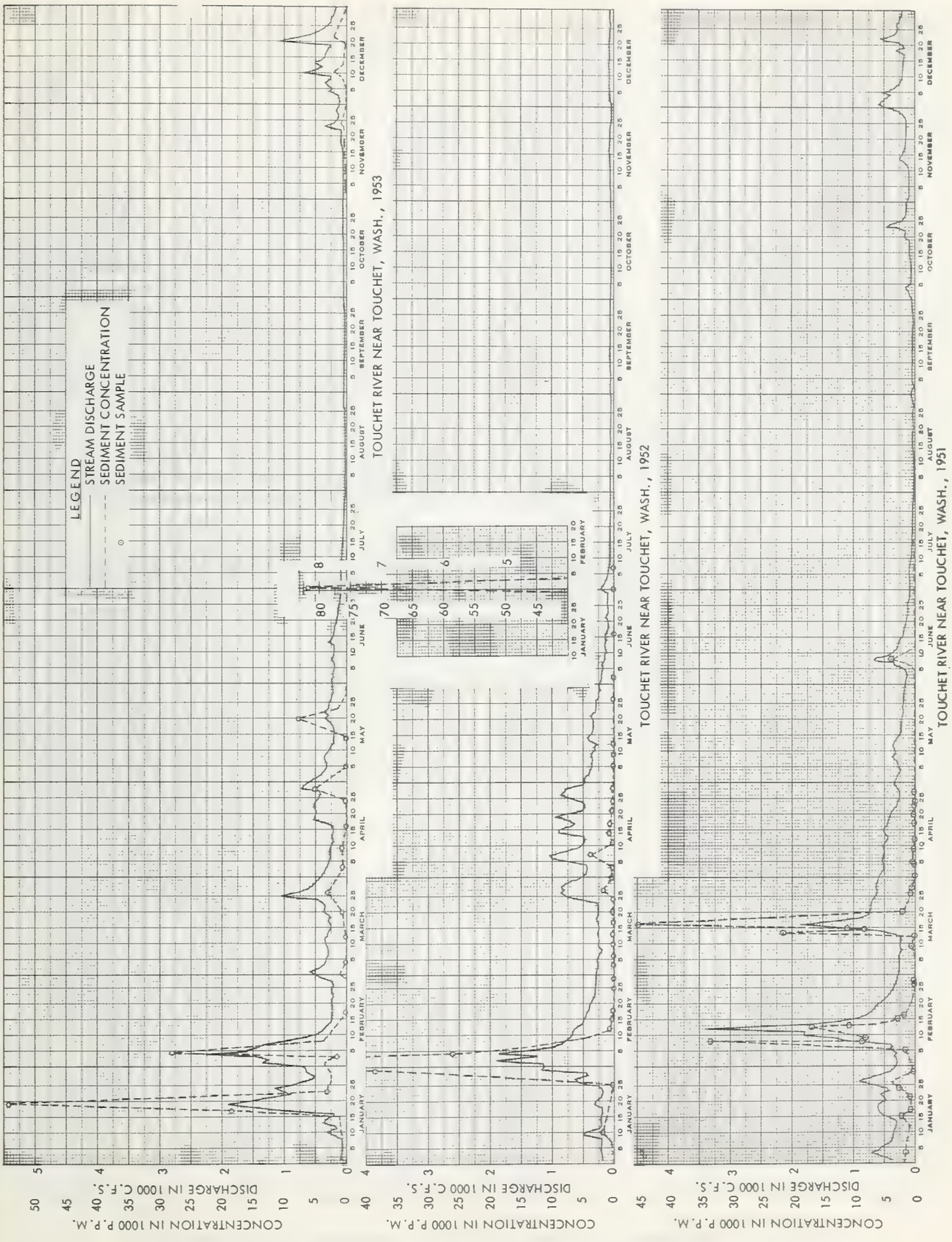


FIGURE 5. — Relationship of sediment concentration to stream discharge, Touchet River.

are as great as 28 feet over the original channel and 18 feet over the original valley floor, as shown on profiles for ranges 4 and 3, respectively. A longitudinal sediment delta profile is shown in figure 16. A large proportion of deposition during the year 1961–62 occurred between ranges 2 and 3 and therefore does not show on the range profiles. The areal extent of deposition as of May 1961 is indicated pictorially by the aerial mosaic view in figures 9 and 10. Approximately two-thirds of the original water surface area upstream of range 1 has been covered with sediment to near normal pool level. Heavy growth of willows and cattails now covers much of this area. This vegetal growth is expected to increase annually and induce further sedimentation during major floods.

The total deposition between date of impoundment, 1953, and August 1961, is computed to be 6,040 acre-feet. The rate of deposition between surveys is shown in table 3.

Data for the 1962 survey were not received in time for analysis and inclusion in this table. The average rate of deposition for the 8-year period 1953–61 is 860 acre-feet. This is equivalent to a rate per year per square mile of drainage area of 0.49 acre-foot. Table 3 also shows a ratio between annual deposition and volume of runoff in acre-feet of about 1 to 500. Comparing the above rate of deposition with the average annual sediment discharge of 1,540 acre-feet estimated from suspended samples indicates that 56 percent of the total sediment load of the Walla Walla River has been deposited in the Walla Walla River arm of the reservoir.

TABLE 3.—*Average annual runoff and sediment accumulation, Walla Walla River*

Period	Runoff	Sediment deposition
	Acre-feet	*Acre-feet*
1953–56	420,400	860
1953–58	441,700	850
1953–60	435,200	740
1953–61	434,900	860

Analyses of deposited sediment were made from samples obtained at four locations (table 4 and figure 8) and show somewhat "soupy"

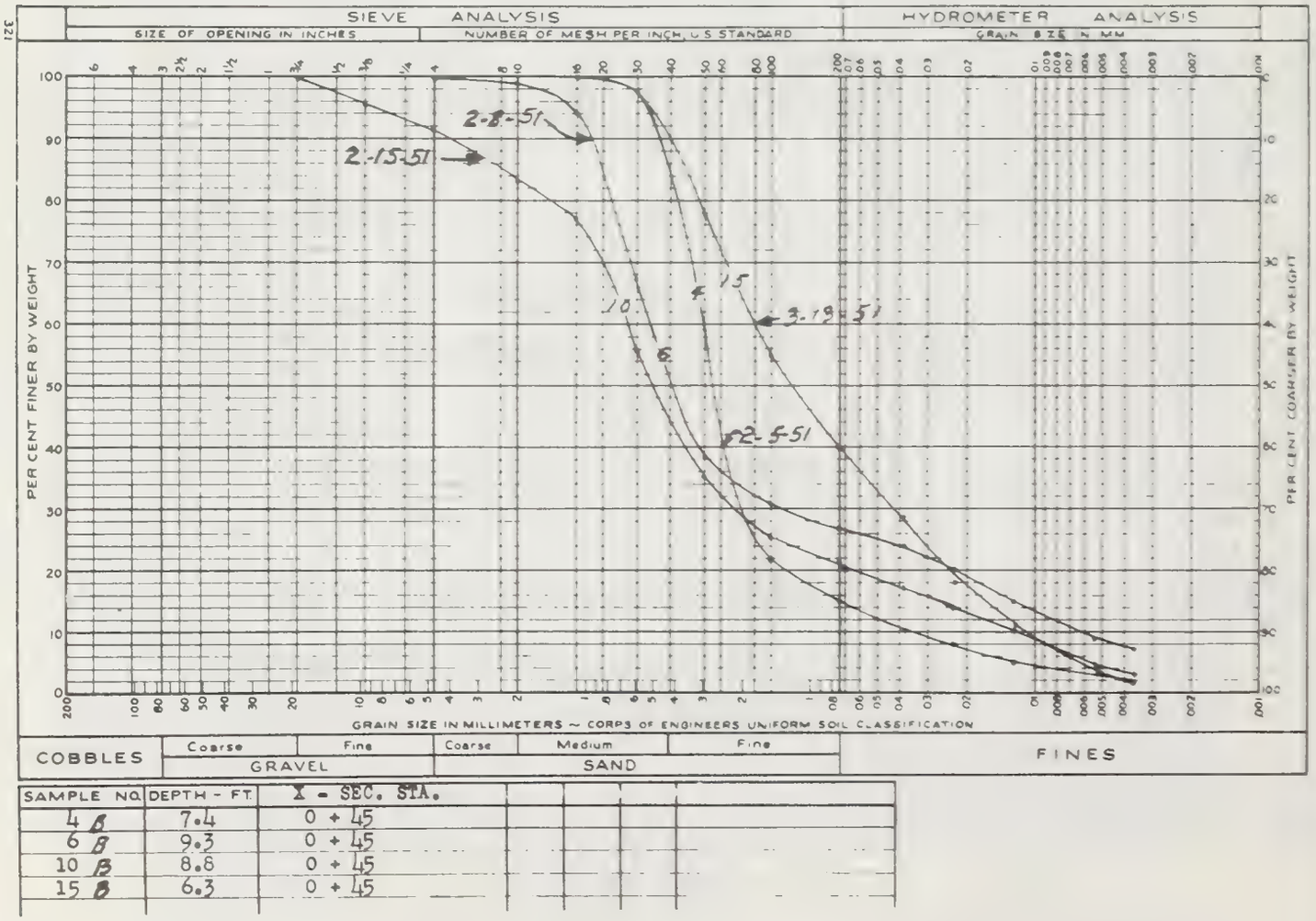
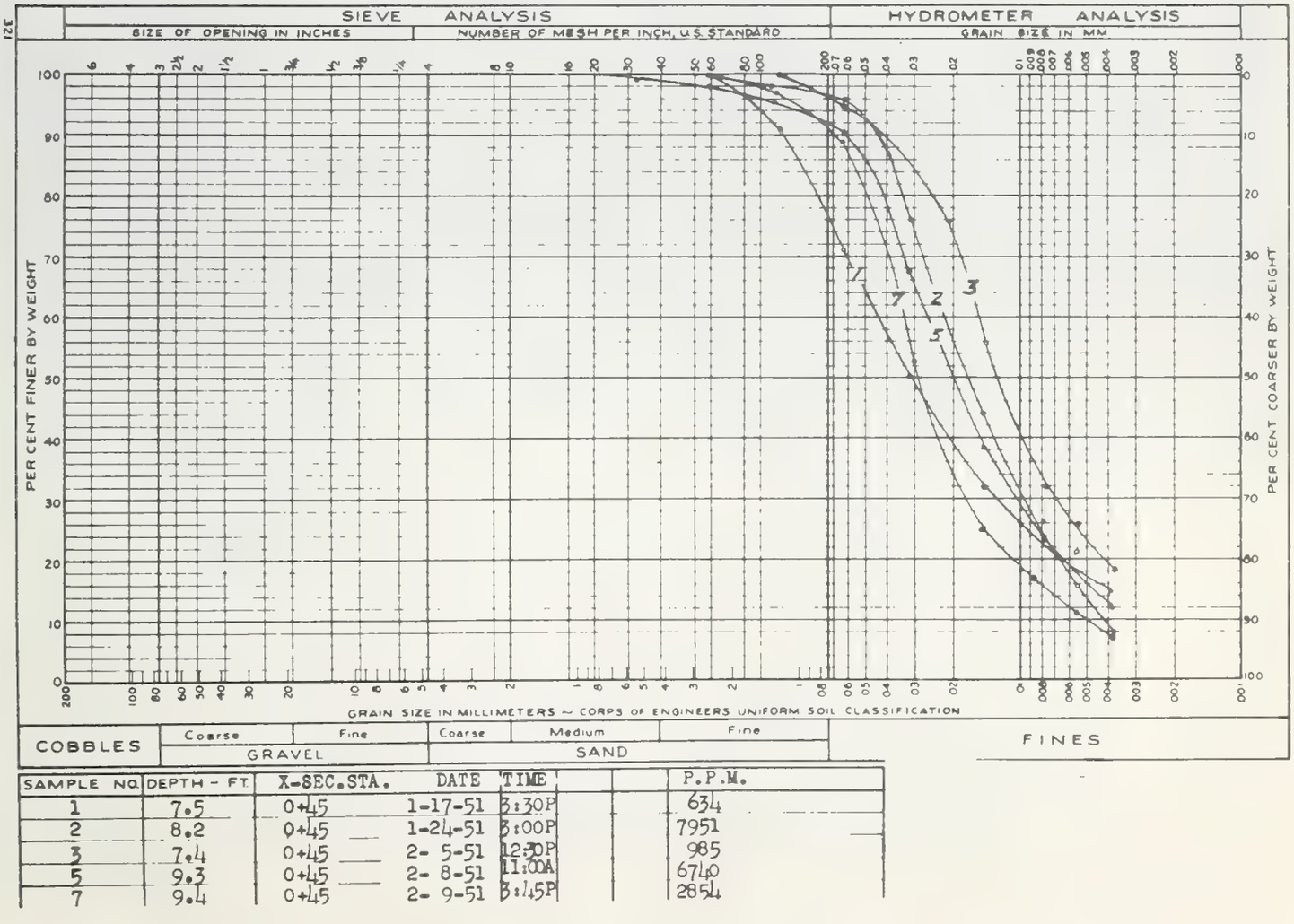


FIGURE 6. — Gradation curves, bed samples, Walla Walla River, Touchet, Wash.

TABLE 4.—*Analyses of delta samples collected in 1960, Walla Walla River arm, McNary Reservoir*

Hole No.	Sample No.	Sample run[1]	Approximate depth of sample	Net weight of material	Moisture content	Wet density	Dry density	Specific gravity
			Feet	*Grams*	*Percent*	*Lb./cu. ft.*	*Lb./cu. ft.*	
P–1	1	1.0–4.0	2.5–3.1	121.3	33.5	112.7	84.3	.....
P–2	1	2.0–4.0	2.5–3.0	111.3	34.8	103.4	76.7	2.53
	2	2.0–4.0	3.0–3.5	122.1	34.8	113.4	84.2	.....
	3	2.0–4.0	3.5–4.0	115.2	36.7	107.0	78.4	.....
	4	4.5–6.0	4.5–5.0	[2] 52.4	36.3	.....	.....	.....
	5	4.5–6.0	5.0–5.5	119.0	39.6	110.6	79.1	2.65
	6	4.5–6.0	5.5–6.0	118.3	38.5	109.9	79.4	.....
P–3	1	1.6–4.0	1.6–2.1	[2] 51.3	31.6	.....	.....	.....
	2	1.6–4.0	2.1–2.6	116.0	39.3	107.8	77.3	.....
	3	1.6–4.0	2.6–3.1	111.8	43.2	103.9	72.4	2.55
	4	4.0–8.0	4.0–4.5	[2] 100.6	59.7	.....	.....	.....
	5	4.0–8.0	4.5–5.0	[2] 99.2	50.7	.....	.....	.....
	6	4.0–8.0	6.5–7.0	108.0	44.5	100.3	69.4	2.52
	7	4.0–8.0	7.0–7.5	117.0	44.0	108.7	75.5	.....
	8	4.0–8.0	7.5–8.0	120.9	43.3	112.3	78.3	.....
P–4	1	2.0–4.0	2.0–2.5	121.0	34.0	112.4	83.8	2.66
	2	2.0–4.0	2.5–3.0	[2] 90.2	39.3	.....	.....	.....
	3	4.0–8.0	5.5–6.0	109.3	45.1	101.6	70.0	.....
	4	4.0–8.0	6.0–6.5	107.8	41.4	100.2	70.7	.....
	5	4.0–8.0	6.5–7.0	111.0	46.2	103.1	70.6	.....
	6	4.0–8.0	7.0–7.5	121.6	35.3	113.0	83.5	2.68
	7	4.0–8.0	7.5–8.0	127.0	39.7	118.0	84.5	.....

[1] Volume of sample cylinder was 67.16 cubic centimeters.
[2] Denotes incomplete sample (no density possible).

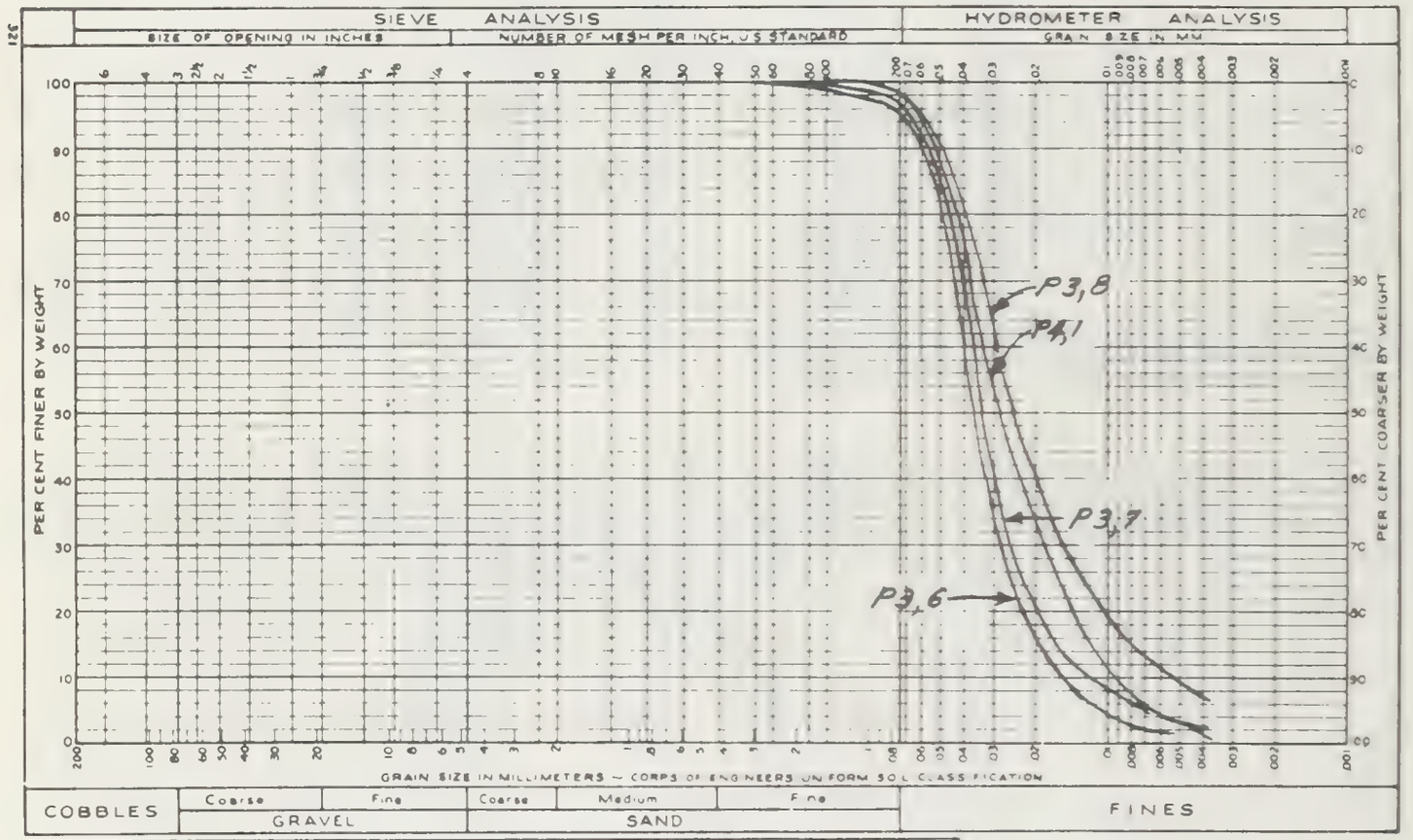


SAMPLE NO.	DEPTH - FT.	X-SEC. STA.	DATE	TIME	P.P.M.
1	7.5	0+45	1-17-51	3:30P	634
2	8.2	0+45	1-24-51	3:00P	7951
3	7.4	0+45	2- 5-51	12:30P	985
5	9.3	0+45	2- 8-51	11:00A	6740
7	9.4	0+45	2- 9-51	3:45P	2854

FIGURE 7. — Gradation curves, suspended samples, Walla Walla River, Touchet, Wash.

condition typical of recent silt deposition. Appreciable further compaction can be expected.

The remaining storage space below normal pool upstream from range 1 is 3,390 acre-feet. Based on the indicated average annual rate of 860 acre-feet of sediment deposition, this area would fill in 3 more years or by the fall of 1965. However, the rate of deposition upstream from range 1 will probably gradually decrease as the deposition approaches the main reservoir and a greater percentage of the sediment is carried out into the main reservoir. If possible variations of sediment load and stream runoff from year to year are considered, the remaining storage space essentially could fill in 1 year if a major flood occurred. In any case, the fore-



SAMPLE NO	DEPTH - FT	CLASSIFICATION	NAT W C	L L	P I	Specific Gravity
P-3, 6	6.5-7.0	SILT	44.5			2.52
P-3, 7	7.0-7.5	SILT	44.0			
P-3, 8	7.5-8.0	SILT	43.3			
P-4, 1	2.0-2.5	SILT	34.0			2.66

FIGURE 8. — Gradation curves, delta samples, Walla Walla River, Wallula, Wash.

going sediment measurements and analyses definitely show that the water storage area in the Walla Walla arm of McNary Reservoir will be essentially filled with sediment in a few more years, except for shallow pools and the channel area.

As yet, there apparently has been no appreciable deposition of sediment downstream from range 1. Large amounts of deposition in this area would be of great concern to the Port District of Walla Walla (fig. 9), as it may affect navigable depths. Wave action is much more pronounced in the main reservoir, and this action is expected to distribute and keep the fine sediment transported by the Walla Walla River from rapidly depositing in the port area.

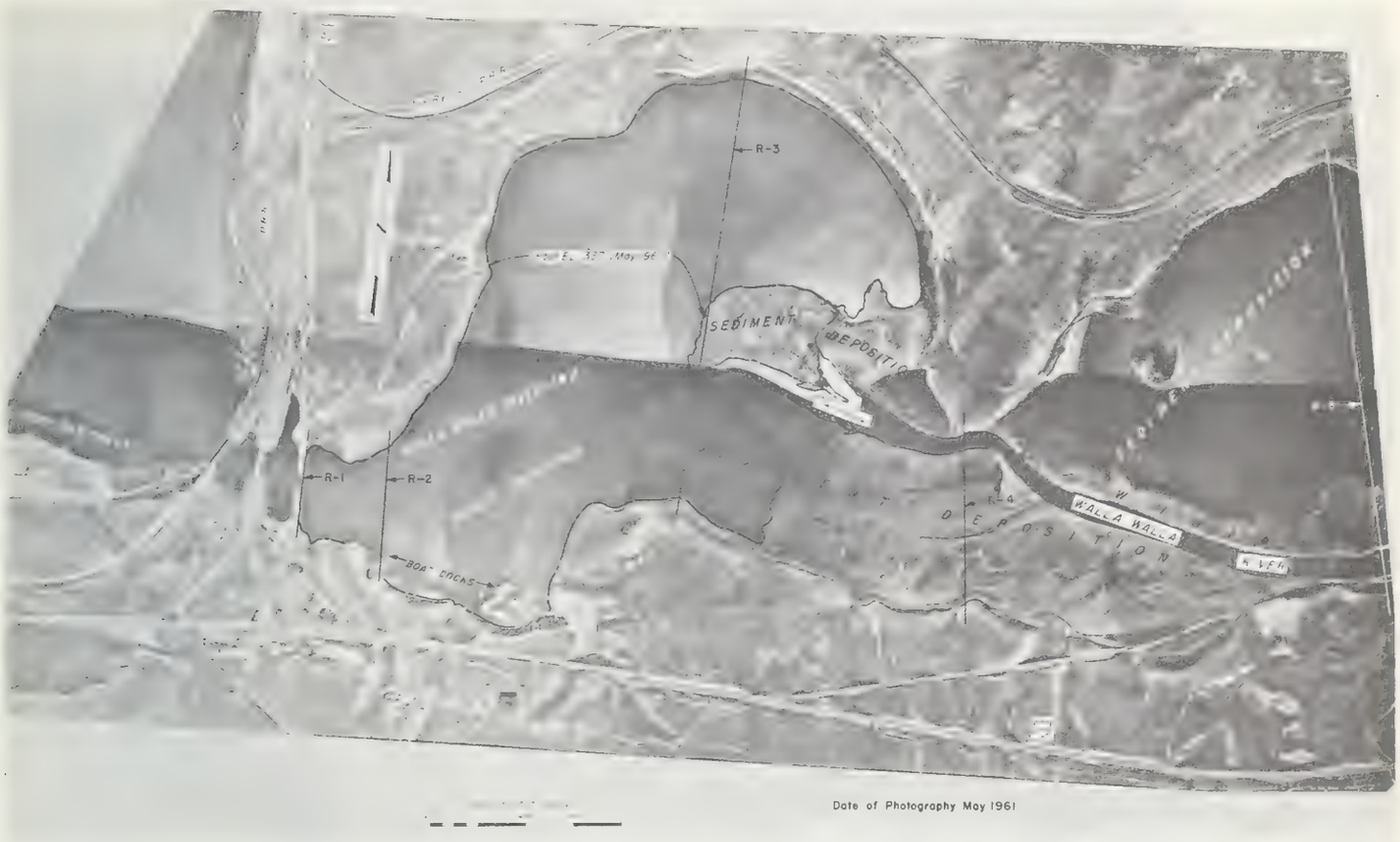


FIGURE 9. — Sediment range layout, Walla Walla River, ranges 1 to 5.

Reservoir Use Development, Walla Walla Arm, McNary Reservoir

Plans Considered

During the early planning for development and management of McNary Reservoir, the Walla Walla arm was selected as an outstanding recreational area. This area was considered very desirable for development as a State park. A preliminary general development plan was prepared (fig. 17). Its chief assets were the apparent ample water area which was well protected from the prevailing southwest wind, access, good boat moorage sites, good beach areas, and scenic views. The principal features planned for the park include a public day use area, boating concession, organized group camp-

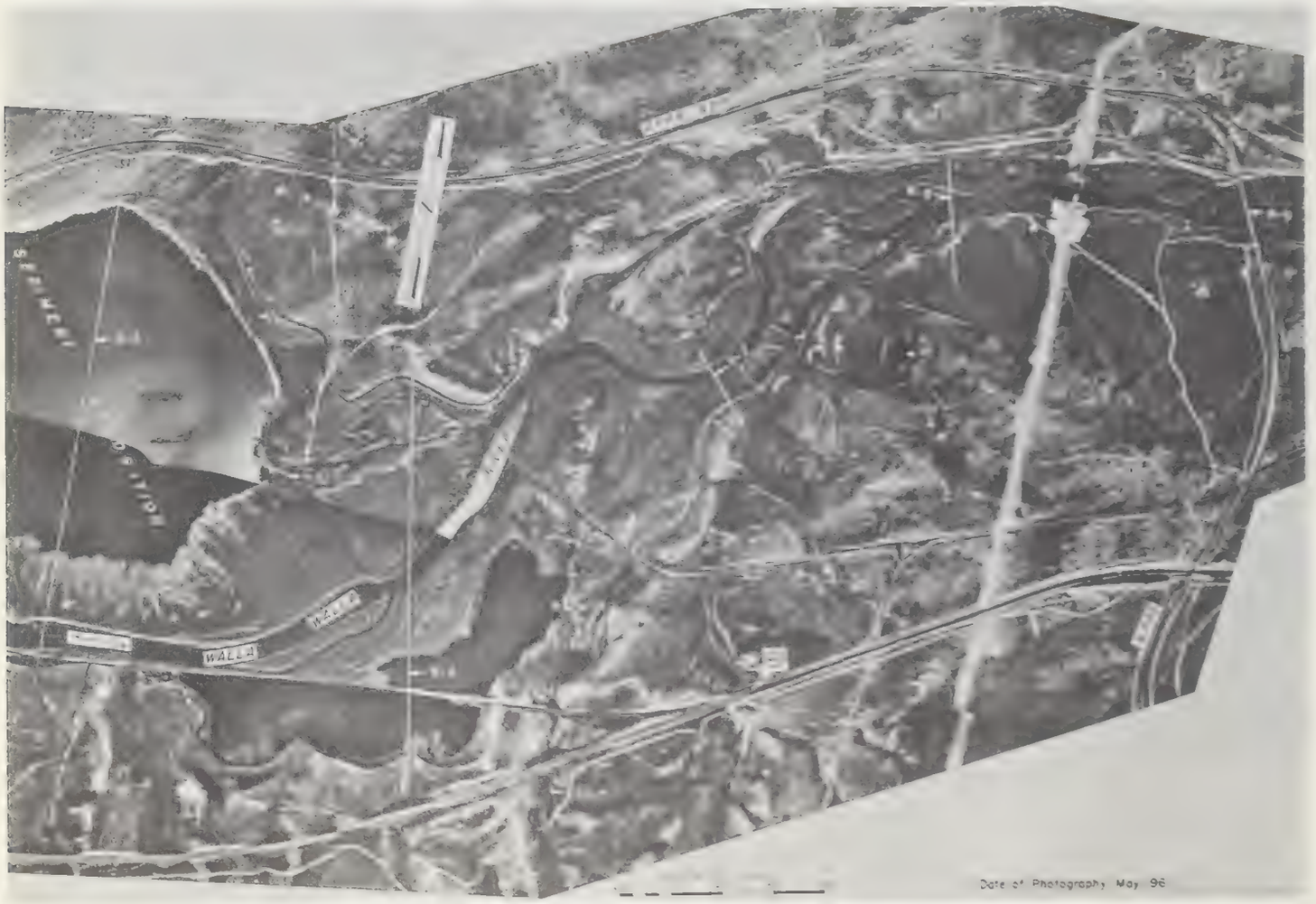


FIGURE 10. — Sediment range layout, Walla Walla River, ranges 5 to 9.

FIGURE 11. — Survey crew and equipment.

FIGURE 12.—Plane table being oriented over cut-in point and boat proceeding to range location to start sounding operations.

ing on the north and south shores, parking areas, launching ramp, and the usual beach and grounds development. Due to the sediment problem, however, recreational development thus far has been limited to a private boat headquarters, public launching ramps, picnic facilities, and wildlife areas. The development of a park has been abandoned by the State of Washington, but there continues to be strong local interest in maintaining use of as much of the area as practicable, particularly as a small boat harbor.

Investigations for Control of Sedimentation

Several plans for protection of the recreation areas and control of sedimentation on the Walla Walla River arm have been considered. The plans consisted of the construction of upstream sediment storage reservoirs on the Walla Walla and Touchet Rivers and the construction of parallel jetties to confine the river in a narrow channel designed to discharge the sediment into the main reservoir, with periodic maintenance dredging as required. Studies of the upstream sediment storage reservoirs proved to be infeasible. Other detailed studies showed the permanent effects of some of the proposed remedial measures could not be completely and accurately evaluated, and the costs were excessive when compared to probable benefits. Jetties for protection of the south shore development are undergoing further study and consideration, with a view to maintaining a small boat harbor.

Future Sediment Program

Important to the future planning and most feasible development of the Walla Walla River arm of McNary Reservoir is accurate and up-to-date information on sedimentation from the Walla Walla River. Because of the relatively short period of existing measurements, the rates of deposition and relationships to runoff are approximate estimates. Therefore, it is planned to continue periodic sediment surveys, including analyses of deposited material. Also, arrangements have been made for the U.S. Geological Survey to obtain regular suspended sediment samples near the mouth of the Walla Walla River.

Sediment deposition at the mouth of tributaries as they enter reservoirs is an important factor in planning reservoir use development at several other locations along the Columbia and Snake Rivers. Locations of such areas along the reservoirs that will be formed by John Day and Lower Monumental Dams are shown in figures 18 and 19, respectively. Information obtained from analyses of sediment measurements on the Walla Walla arm of McNary Reservoir has been useful in predicting amount and distribution of sediment at these locations. However, more information is needed and a program of obtaining regular suspended samples on the tributary streams has been instituted in cooperation with the U.S. Geological Survey.

Conclusions

The Walla Walla arm of the McNary Reservoir is rapidly filling with sediment. Based on five sediment surveys since 1953, the average annual rate of deposition is 860 acre-feet. The annual rate of deposition per square mile of Walla Walla River drainage area is 0.49 acre-foot. The estimated average annual sediment discharge of the Walla Walla River is 1,540 acre-feet.

The sediment deposition has greatly altered the original plans for recreational development of the area. Additional information and studies are required to determine the feasibility of providing protection and maintenance of a small boat harbor on the south shore, and the effects of port development.

A program of sediment surveys and suspended samples will continue in order to define more accurately rates and locations of sediment deposition in the Walla Walla arm and at other locations in reservoirs along the Columbia and Snake Rivers.

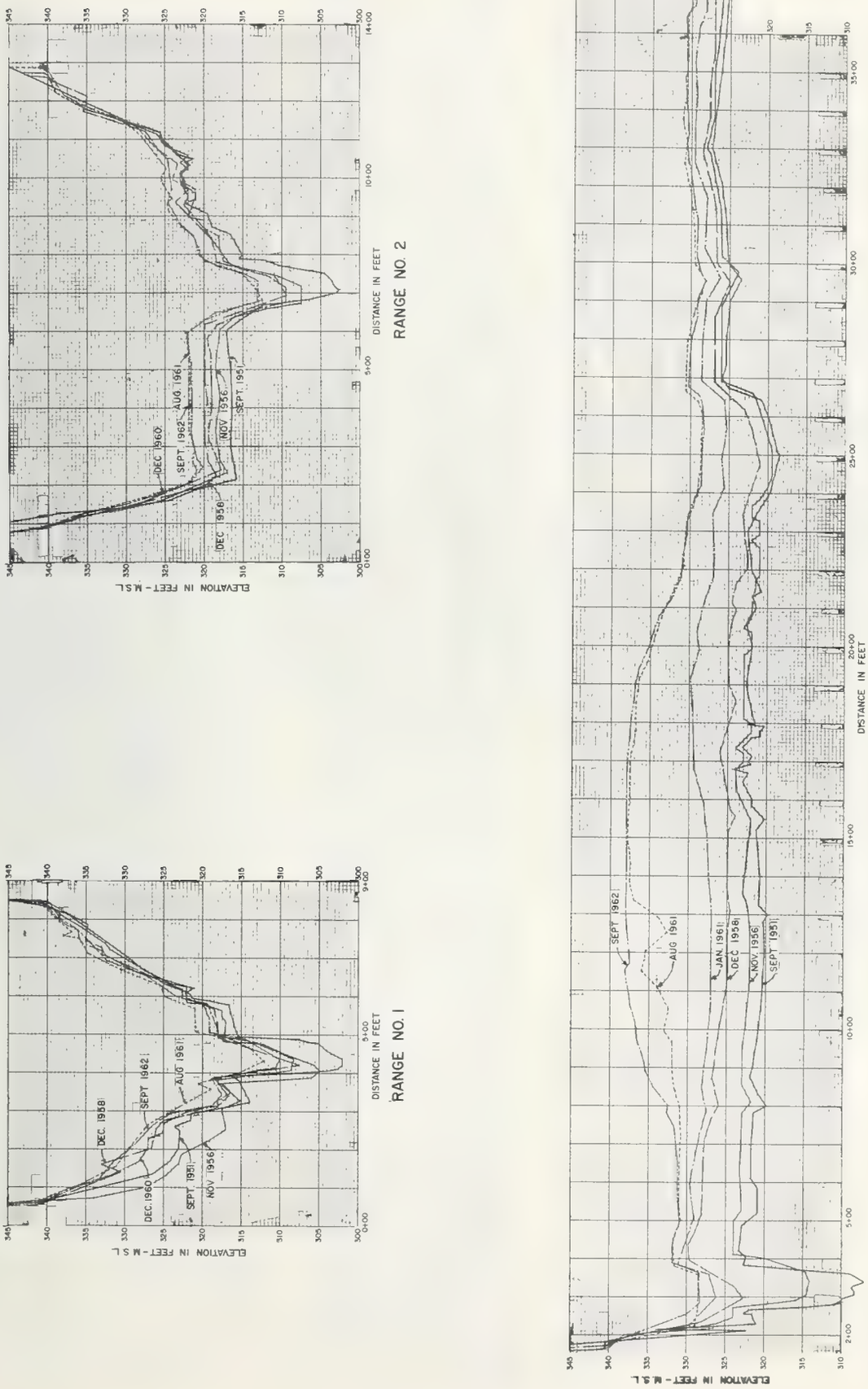
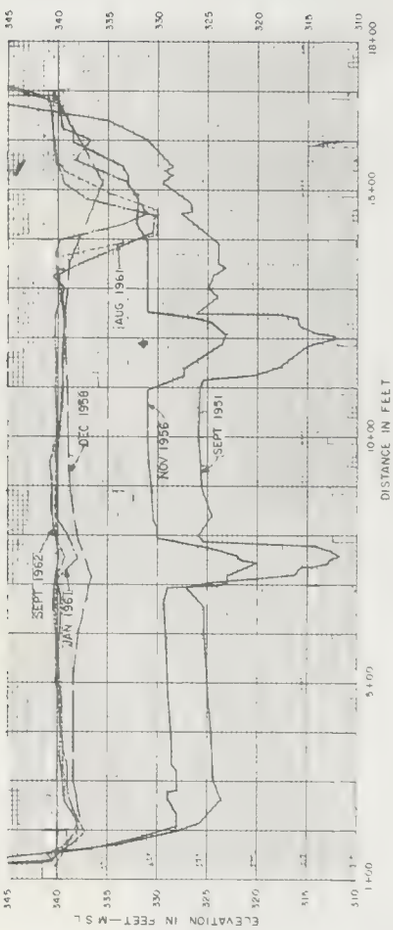
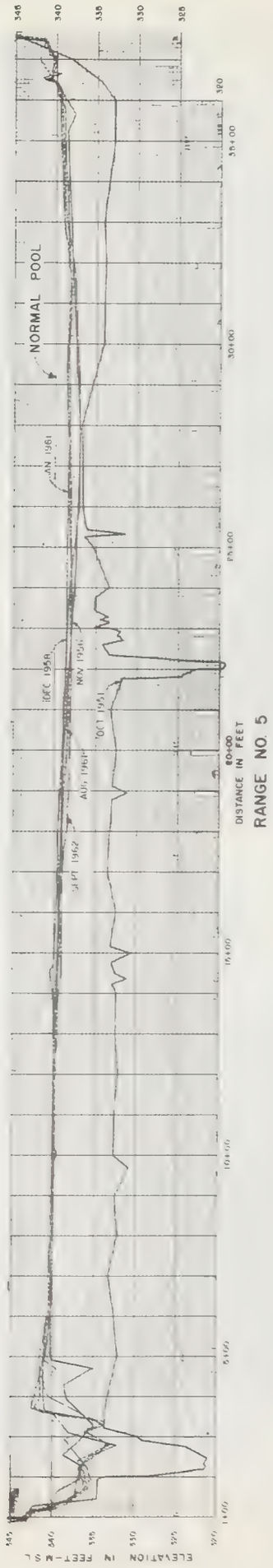


FIGURE 13. — Sediment range profiles 1 to 3, Walla Walla River.





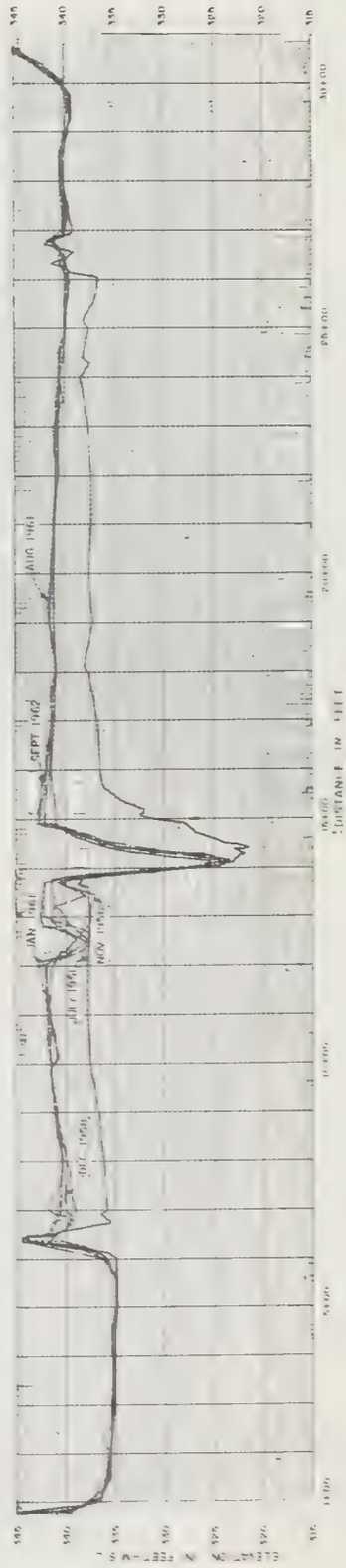
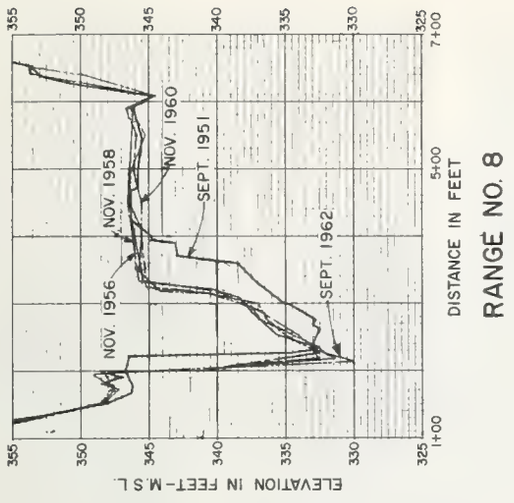
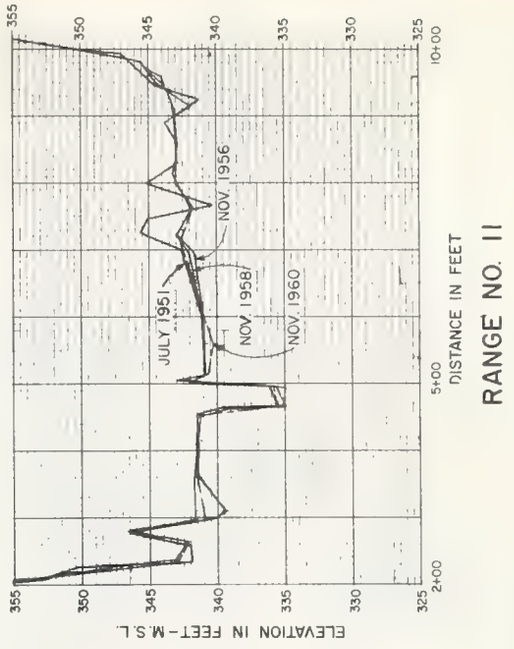
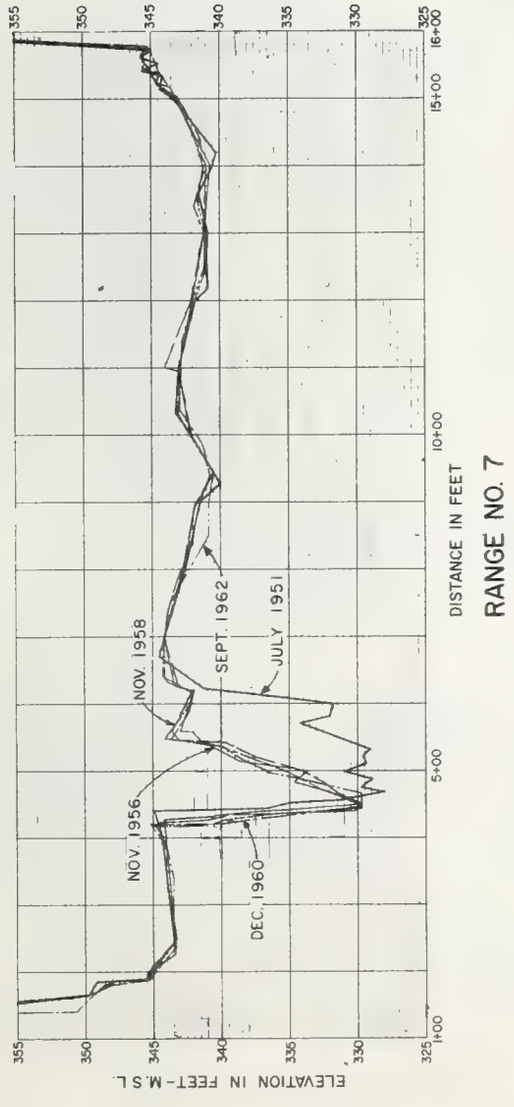
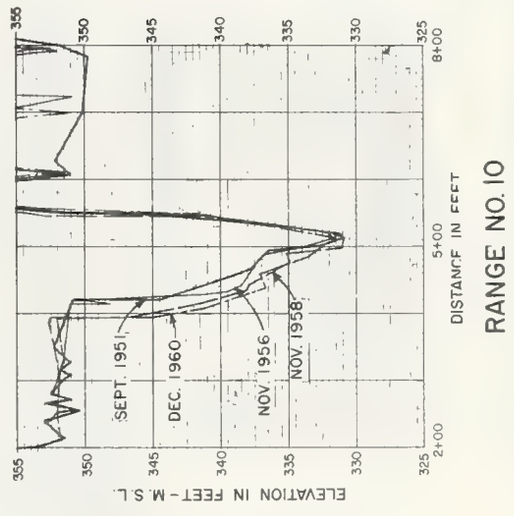


Figure 14. — Sediment range profiles 4 to 6, Walla Walla River.









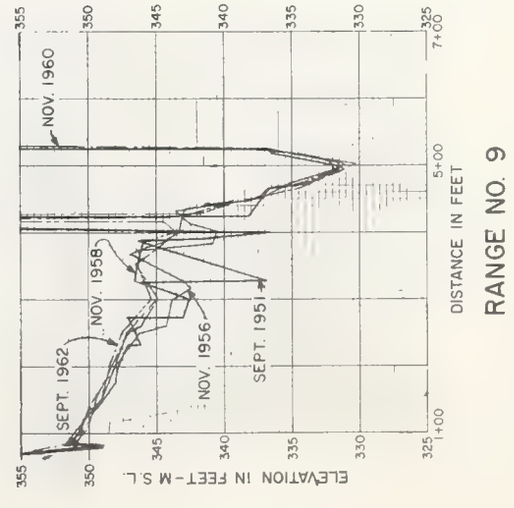


FIGURE 15. — Sediment range profiles 7 to 11, Walla Walla River.

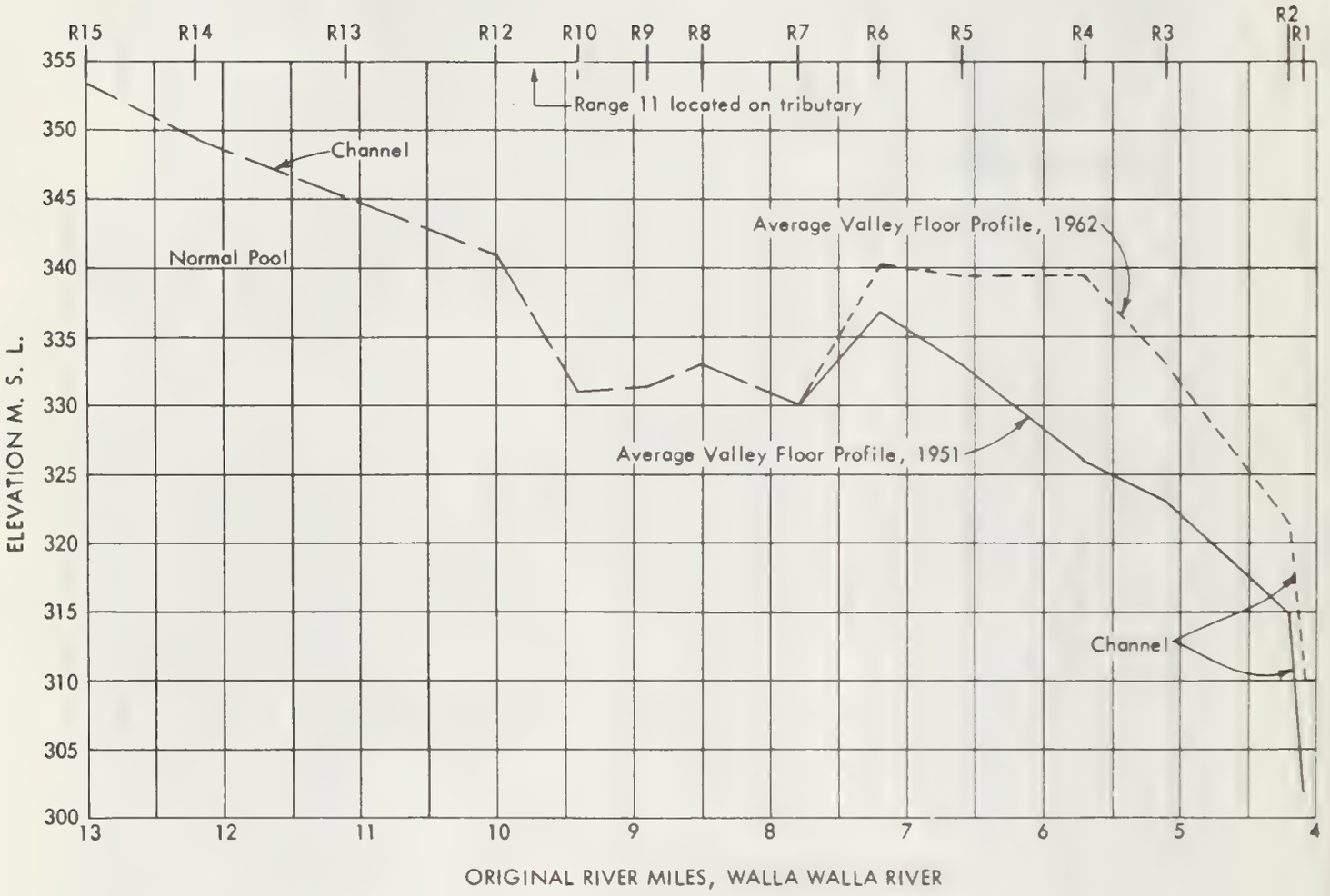


FIGURE 16. — Longitudinal profile sediment ranges, Walla Walla River.

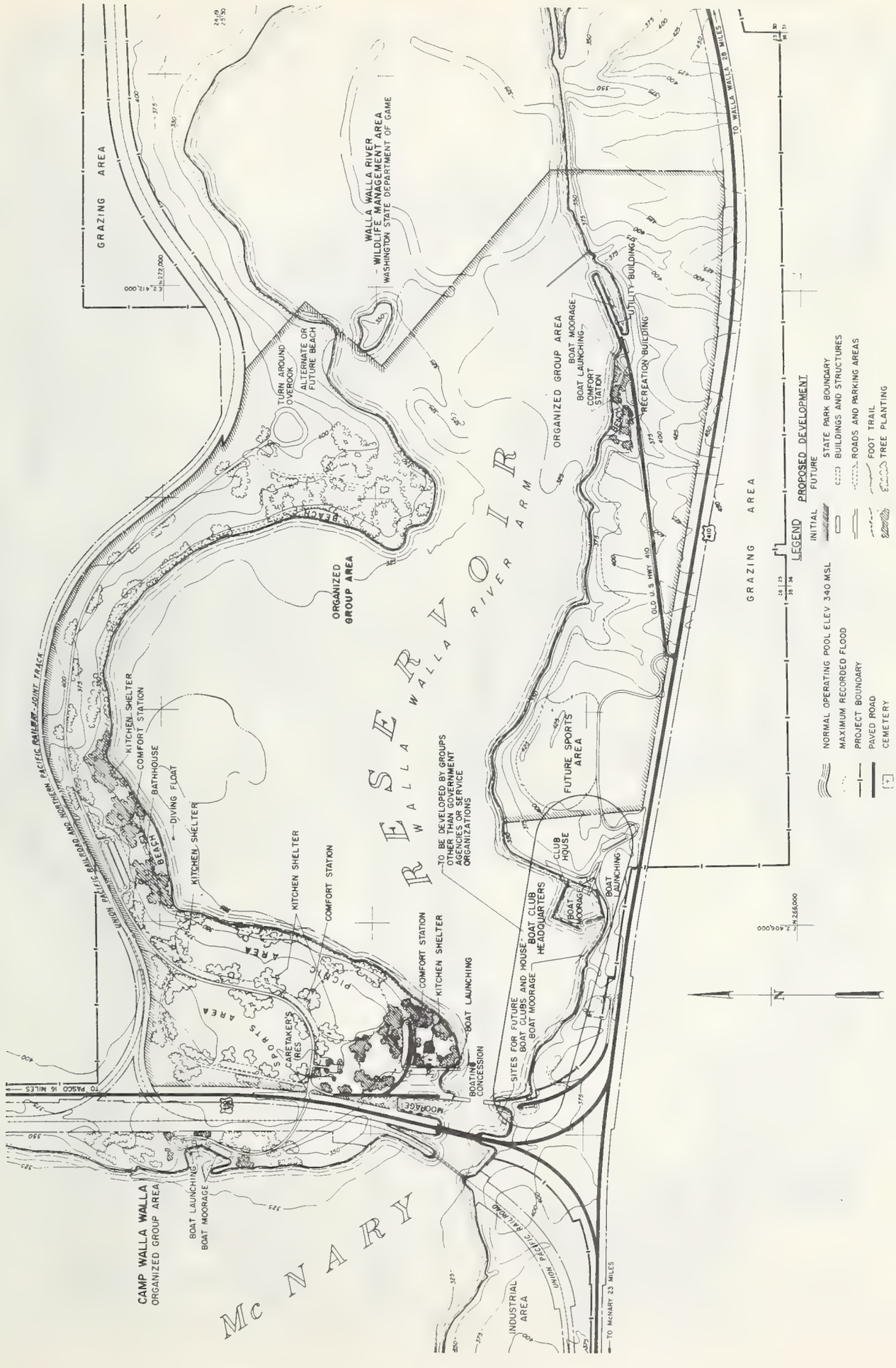


FIGURE 17.—Preliminary development plan for State park.

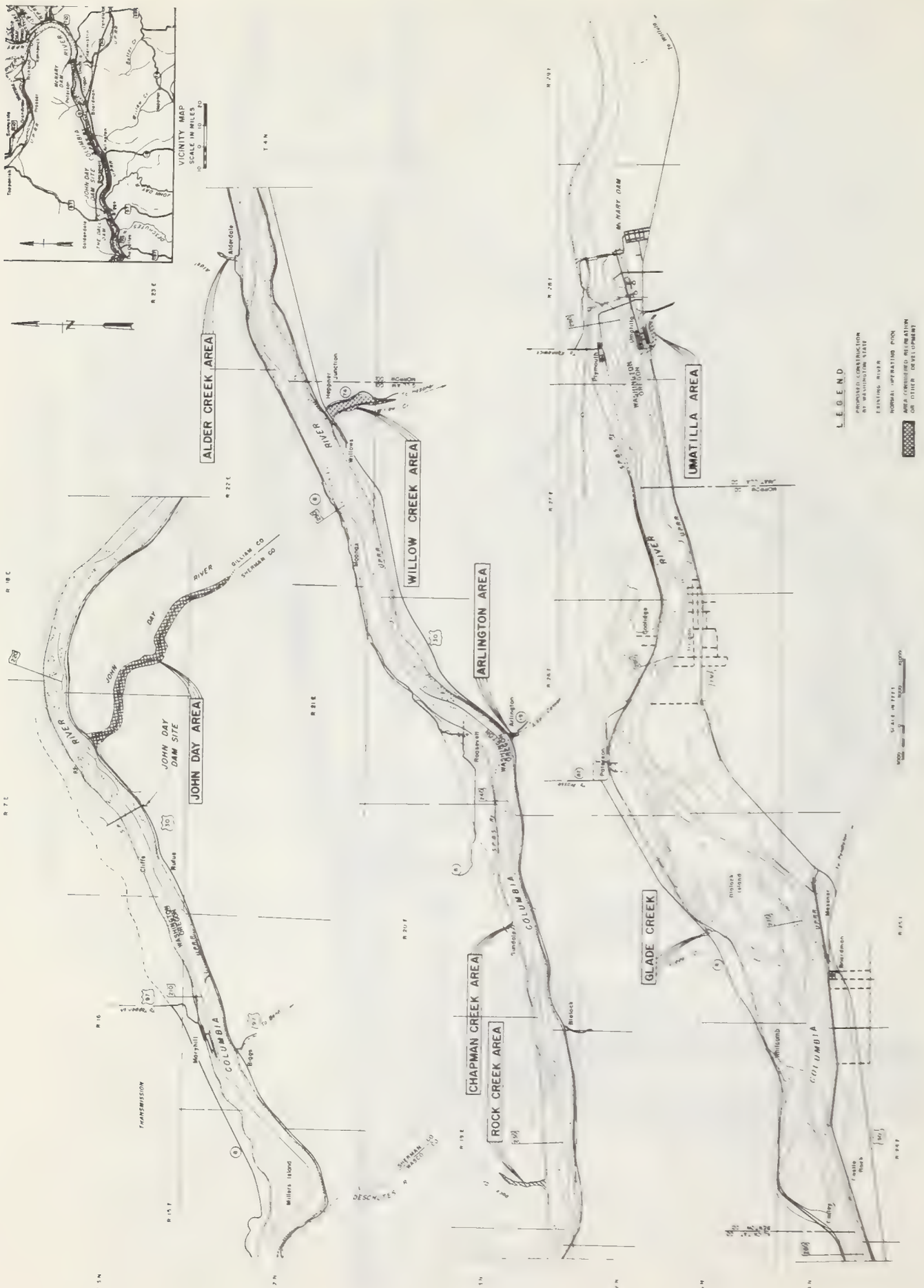


FIGURE 18. — John Day Reservoir map.

FIGURE 19. — Lower Monumental Reservoir map.

CHARACTERISTICS OF SEDIMENT DEPOSITS ABOVE CHANNEL STRUCTURES IN POLACCA WASH, ARIZ.

[Paper No. 80]

By R. F. HADLEY, *geologist, U.S. Geological Survey, Denver*

Abstract

Deposition caused by the construction of dams on Polacca and Wepo Washes since 1945 is estimated at 7,500 acre-feet. Longitudinal profiles were surveyed above each dam on Polacca Wash. The present channel gradients on the sediment deposits varies from 0.0005 ft./ft. to 0.0037 ft./ft. The gradients of these channels prior to dam construction ranged from 0.0040 ft./ft. to 0.0058 ft./ft.

Of the total sediment deposited behind the structure approximately 8 percent is located above the spillway level and aggradation was observed some distance above the anticipated influence of the dams.

Introduction

Many of the alluvial valleys in southwestern United States have been trenched by deep arroyos in the past 70 years. Many others have just begun to degrade; a few have retained a continuous, relatively flat valley floor or are aggrading. These various stages of erosion and alluviation often can be observed in contiguous drainage basins. Conservation measures, such as construction of dams and water-spreading dikes, have been initiated in some trenched valleys in an effort to restore the usefulness of the valley floor for grazing and floodwater farming by promoting aggradation of the arroyo.

The primary objective of this report is to describe the extent of channel and flood plain aggradation induced by dams and the effect of these deposits on the longitudinal profile of the arroyo. The study was done in the Polacca Wash drainage basin in northeastern Arizona (fig. 1).

Polacca Wash and its principal tributaries, Wepo Wash and Keam's Canyon Wash, are typical of the many streams that drain Black Mesa in the Navajo and Hopi Indian Reservations. In the upper third of the basin the streams are narrow and are confined in deeply incised canyons. Near the village of Polacca, they become wider, and flow in broad, relatively flat valleys.

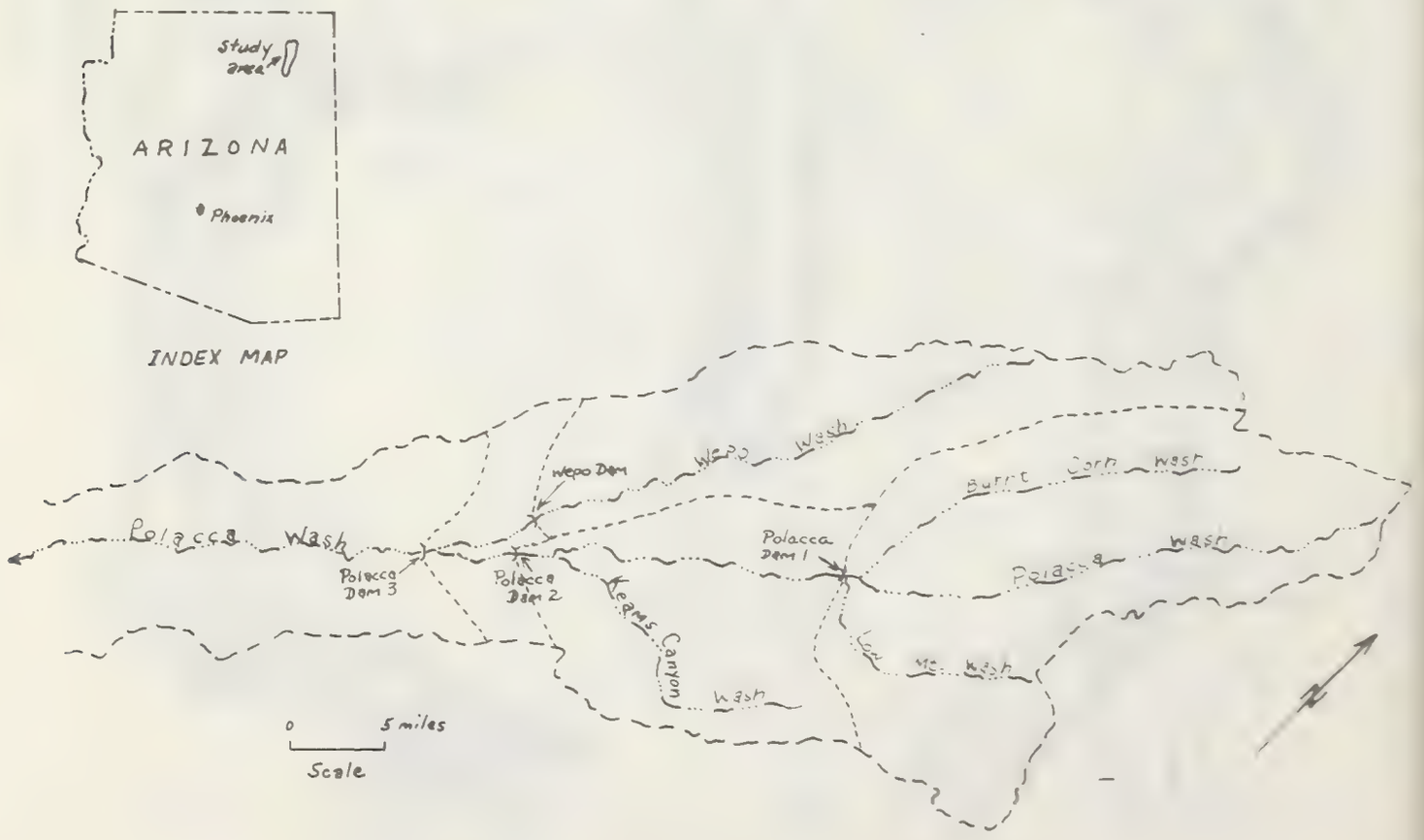


FIGURE 1. — Polacca Wash basin, showing location of dams.

Windblown sand on the valley floors obscure many of the smaller tributaries. In the central and lower reaches of the drainage basin the tributaries are generally aggraded, forming alluvial fans that obliterate the confluence with main channels. For most of its length the present Polacca Wash channel is deeply entrenched in the valley alluvium.

Deposition Caused by Channel Structures

The amount of sediment deposited in the channels and on the valley floors of Polacca and Wepo Washes caused by construction of the diversion dams and waterspreaders was calculated from surveys of the longitudinal profile and valley cross sections and mapping on aerial photographs. Limitations must be placed on the precision of these calculations because of the lack of complete data on the original profile of the streambed and shape of the channel. However, measurements of the channel and valley downstream from the dam probably provide an accurate representation of the original reservoir contours above the dam. Also, by extending the longitudinal profile from some point downstream from the dam to a point where the extension intersects the channel floor above the reservoir deposits, the original profile is closely approximated. The channel profile and valley shape do not vary greatly in a distance equal to that affected by a dam. Using the reconstructions to provide data on valley shape and slope, the volume of sediment was calculated.

Between about 1900 and 1945 the channel of Polacca Wash eroded to a depth of about 40 feet in the reach near the village of Polacca. In the years 1945 to 1953 three dams were constructed on Polacca Wash in this reach and another near the mouth of Wepo Wash. (See fig. 1.) The purpose of these structures is to divert flood flows onto the valley floor for irrigation and to promote deposition of sediment in the arroyo.

Polacca Dam 2 was constructed in 1945 and is located about 4 miles upstream from the mouth of Wepo Wash. The crest of the dam is 47 feet above the bed of the arroyo, which is about 40 feet deep in this reach. Sediment has almost completely filled the channel behind the dam for a distance of 4,000 feet. About 3,000 acre-feet of sediment have been deposited in the reservoir since 1945 at an annual rate of about 0.5 acre-foot per square mile. Since 1953 storage in Polacca Dam 1 has reduced the sedimentation of the reservoir of Polacca Dam 2.

About one-quarter mile below the mouth of Wepo Wash is Polacca Dam 3, constructed in 1946. At this site Polacca Wash is entrenched 38 feet into the valley floor. The crest of the dam is 48 feet above the bottom of the wash and subsequent deposition of sediment behind this barrier completely filled Polacca Wash for 9,030 feet upstream. The maximum depth to which sediment has accumulated at the dam is 38 feet. The average thickness of sediment behind the dam is 21 feet (table 1), and the total volume of sediment deposited back of Polacca Dam 3 is 1,090 acre-feet. The average annual rate of sediment accumulation is 0.8 acre-feet per square mile for the period 1946-57. Polacca Dam 3 has served primarily to divert runoff to the waterspreading area along the south bank of Polacca Wash below the damsite, and to prevent further deepening of Polacca Wash. This dam caught all the overflow from Polacca Dam 2 before construction of a training dike next to the arroyo bank and any uncontrolled flow from Wepo Wash. The overbank deposition has raised the level of the valley floor 3 feet in the reservoir area and channel deposits extend to an elevation 15 feet above the spillway.

TABLE 1.—*Estimates of deposition caused by construction of dams on Polacca and Wepo Washes, Ariz.*

Name of structure	Surface area of reservoir	Maximum thickness of deposition	Average thickness of deposition	Volume of sediment deposition	Area of water-spreading	Volume of sediment deposition on waterspreading area[1]	Drainage area
	Acres	*Feet*	*Feet*	*Acre-feet*	*Acres*	*Acre-feet*	*Square miles*
Polacca Dam 1	180	15	11	1,900	260	80	338
Polacca Dam 2	133	38	23	3,000	586	290	511
Polacca Dam 3	53	38	21	1,090	1,710	210	757
Main Wepo Dam	26	15	7	190	920	690	197

[1]Estimates for deposition on waterspreading area are probably less than actual amounts because much of the sediment has been disturbed in repair of training dikes.

About 14 miles upstream from Polacca a third dam was constructed on the main channel and has been designated Polacca Dam 1. This dam was completed in 1953 and is located just below the mouth of Polacca Wash with Burnt Corn Wash and Low Mountain Draw (fig. 1). The arroyo has been completely filled above the structure for a distance of 2,300 feet, and channel deposits extend upstream for 7,500 feet to the mouth of Burnt Corn Wash.

The channel downstream from Polacca Dam 1 is a rectangular-shaped arroyo and most of the storage in the reservoir was confined to a simi-

lar channel upstream. The maximum thickness of deposition in the reservoir is estimated to be 15 feet and the average is about 11 feet. The volume of sediment deposition in the reservoir is calculated to be 1,900 acre-feet, and the annual rate of deposition to be 1.4 acre-feet per square mile. The drainage area above Polacca Dam 1 is about 340 square miles and, although it is not certain how much area is controlled by structures, an inspection of part of the basin, which is accessible by roads, revealed no appreciable diversion or storage.

On Wepo Wash the main dam was constructed in 1944 about 1¼ miles above the mouth. The crest of the dam is 25 feet above the floor of the Wash and has caused channel deposition for a distance of 5,360 feet upstream. At the dam, sediment has accumulated to a maximum depth of 15 feet. The total volume of sediment accumulated above the main Wepo Dam is 190 acre-feet, with an average thickness of 7 feet (table 1). Downstream from the main dam are two training dikes that divert overflow from the dam away from the main channel of Wepo Wash. Much of the sediment retained in the waterspreading areas below the main Wepo Dam has been utilized to construct additional training dikes, thus making any determination of the total amount of sediment deposited uncertain. The average annual rate of sediment accumulation above the main Wepo Dam is 0.1 acre-foot per square mile for the period 1944–57.

Some comparisons may be made between the suspended-sediment load carried by the Little Colorado River, as measured near Grand Falls, Ariz., and the estimates of sediment trapped by the structures on Polacca Wash. The Little Colorado River basin has a drainage area of 21,200 square miles above Grand Falls and available records of suspended-sediment load were complete there for 9 years at the time of the study (1933, 1948–55). The suspended-sediment loads range from 1,000 to 13,900 acre-feet annually, with an average for the 9-year period of 5,640 acre-feet. Distributed over the drainage basin, this figure represents an average annual sediment yield of about 0.30 acre-foot per square mile.

Since 1945, when the first dam was constructed, the structures in Polacca Wash basin, including waterspreading systems, have trapped approximately 7,500 acre-feet of sediment. The drainage area above these structures comprises approximately 950 square miles. The annual sediment yield per unit of drainage basin has been discussed previously and, because the structures were completed at different times, a mean rate of sediment yield, 0.7 acre-foot per square mile, will be used for comparison with the little Colorado River.

The data seem to indicate that the Polacca Wash area has a higher unit rate of sediment yield and consequently more severe erosion than other areas within the Little Colorado River basin, with a mean annual rate of sediment yield nearly three times that of the river at Grand Falls. It has been shown in other studies,[1] however, that in similar drainage basins the sediment yield decreases with increase in size of drainage area. Therefore, areas of similar size and geology in the Little Colorado River basin may have sediment yields comparable to Polacca Wash.

Relation of Channel Slope to Deposition

Gradients were measured on the deposits upstream from the dams to the apparent limit of deposition. Results of these profile surveys and comparisons with approximations of the original channel profiles are shown in table 2. The channel above Polacca Dam 1 has been filled to spillway level for a distance of 1,000 feet above the dam and has a gradient of 0.0005 ft./ft. and a second wave of sediment with a steep front 3½ feet high is encroaching on the dam. Therefore, two values for the gradient are shown in table 2. The arroyo in Polacca Valley has been aggraded to an elevation higher than spillway above the three dams that have been constructed and overbank deposition has raised the valley floor in some places. The appearance of the channel and flood plain upstream from each of these dams is quite similar to valleys in the Polacca Wash area that are aggrading naturally.

TABLE 2.—*Comparison of original and present channel gradients above Polacca Wash diversion dams*

Structure	Original gradient[1]	Present gradient
	Ft./ft.	*Ft./ft.*
Polacca Dam 1	0.0040	0.0005–0.0037
Polacca Dam 2	.0041	.0016
Polacca Dam 3	.0058	.0010

[1] Original gradients as determined from survey of the channel and reconstruction of profile.

In other studies of the slope of reservoir deposits similar results were found. In the Mexican Springs experimental area near Gallup, N. Mex., several dams were constructed in 1936 to study the effects of waterspreading. Topography of the arroyo channel and valley floor were mapped in detail at that time by the Soil Conservation Service. The gradient of the arroyo floor ranged from 0.0071 ft./ft. to 0.0105 ft./ft. and averaged 0.0075 ft./ft. In 1949 when

[1] HADLEY, R. F., and SCHUMM, S. A. SEDIMENT SOURCES AND DRAINAGE BASIN CHARACTERISTICS IN UPPER CHEYENNE RIVER BASIN. U.S. Geol. Survey Water-Supply Paper 1531-B: 137–198. 1961.

the arroyos had been completely aggraded behind the dams, the gradients of the alluvial fills were measured by the Geological Survey.[2] The gradients ranged from 0.0025 ft./ft. to 0.0044 ft./ft. and averaged 0.0037 ft./ft.

The Bureau of Reclamation conducted a study in 1952 and 1953 of the amount and location of sediment behind Zion Dam on the Little Colorado River near St. Johns, Ariz.[3] The gradient of the original channel from the dam upstream to the elevation of spillway crest, as determined by borings, ranged from 0.00070 ft./ft. to 0.00148 ft./ft. and averaged 0.00123 ft./ft. The gradient of the channel on the alluvial fill ranges from 0.0053 ft./ft. to 0.00070 ft./ft. and averages 0.00058 ft./ft.

The gradient of the original channel in each of the areas described was much steeper than the channel that now flows at or near the level of the valley floor. The data also indicate that the slope is related, in some degree, to the size of drainage area above the point of measurement. In figure 2 the relation between stream

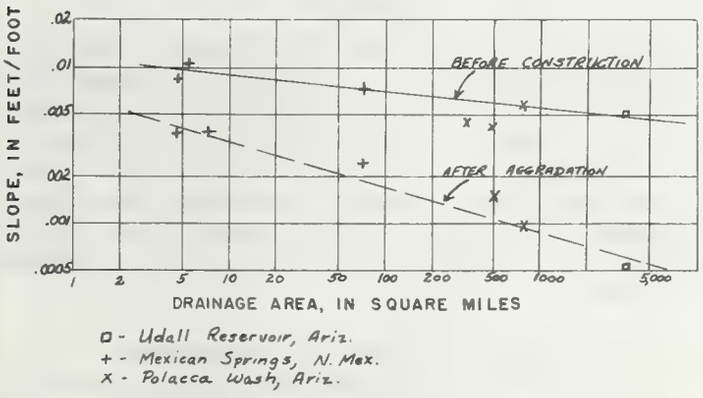


FIGURE 2. — Relation of drainage area to channel slope for several areas in Arizona and New Mexico.

gradient and drainage area is shown for the areas in Polacca Wash, Mexican Springs, and Little Colorado River. The longitudinal profiles of these channels prior to construction of the dams probably were representative of ephemeral channels in equilibrium or quasi-equilibrium. A similar relation for uncontrolled ephemeral channels is shown for New Mexico.[4]

[2] PETERSON, H. V., and SNYDER, C. T. SEDIMENT DEPOSITION IN THE MEXICAN SPRINGS EXPERIMENT STATION, NAVAJO INDIAN RESERVATION, N. MEX. U.S. Geol. Survey Memo. Rpt. 1949.

[3] U.S. BUREAU OF RECLAMATION. REPORT ON SEDIMENT DEPOSITION ABOVE ZION DAM, LITTLE COLORADO RIVER BASIN, ARIZ. 1955. [Processed.]

[4] LEOPOLD, L. B., and MILLER, J. P. EPHEMERAL STREAMS — HYDRAULIC FACTORS AND THEIR RELATION TO THE DRAINAGE NET. U.S. Geol. Survey Prof. Paper 282-A: 27. 1957.

The gradients of the channels developed on the alluvial fills are also shown in figure 2. The gradient at each station is less than the original gradient and decreases with an increase in size of drainage area. The longitudinal profile is dependent primarily on median size of alluvial material, density of vegetational cover, or any number of hydraulic factors.

The relation between channel slope and median diameter of bed material in the aggraded channels is presented in figure 3. There is some

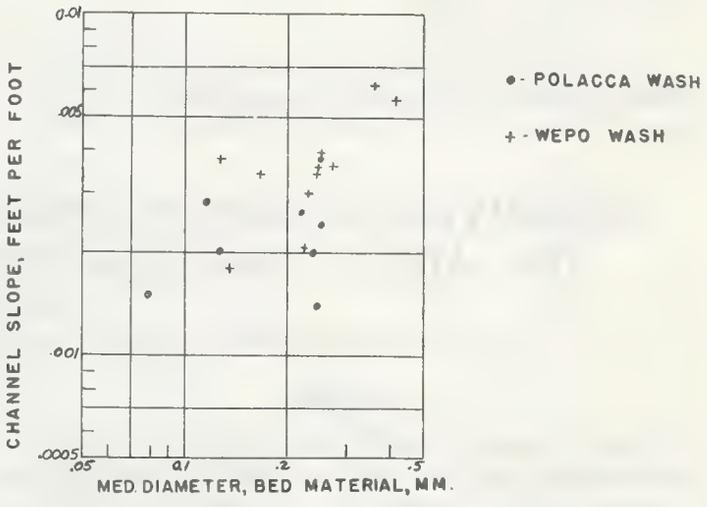


FIGURE 3. — Relation of median diameter of bed material to channel slope.

indication that an increase in size of bed material is accompanied by an increase in channel slope. The narrow range of median diameters, however, prevents defining the relationship more precisely.

Location of Reservoir Deposits

The location of sediment deposits in the reservoir area and channel above a dam are of prime importance to engineers responsible for design and allocation of space for dead storage of sediment. The effect of an artificial base level produced by the construction of a dam will result in marked changes in the stream channel and flood plain for some distance upstream above the elevation of the spillway, which is the new control.

In the studies of the Polacca Wash dams estimates were made of the volume of sediment trapped and of the original profile of the channel reach within the influence of the dam. From these data we may arrive at some approximate figures for the quantity of sediment that has been deposited above the elevation of the spillway control and to what elevation above the spillway deposition occurs.

Estimates of deposition above the present

spillway elevation were made from reservoir cross sections and aerial photographs. The results of these computations for the Polacca Wash structures are shown in table 3.

TABLE 3.—*Location of reservoir sediment above Polacca Wash diversion dams*

Dam	Total sediment trapped	Sediment deposited above spillway level	Percentage of total	Height of deposition above spillway
	Acre-feet	*Acre-feet*		*Feet*
Polacca Dam 1	1,900	100	5	12.5
Polacca Dam 2	3,000	275	9	13
Polacca Dam 3	1,090	108	10	15

The data in table 3 show that of the total sediment trapped in the three reservoirs, 5 to 10 percent lies above present spillway elevation and also that deposition occurs from 12.5 to 15 feet above the spillway elevation. In the study made by the U.S. Bureau of Reclamation on Zion Dam near St. Johns, Ariz., it was concluded that about 10 percent of the reservoir sediment lies above the present spillway crest elevation.

In summary, we may conclude that a change in stream regimen affected by a rising base level such as a dam will reduce the channel slope and cause aggradation upstream to a higher elevation than that of the channel control. The extent of this deposition may be affected by valley width, channel slope, particle size of available material, and influence of riparian vegetation. The relative importance of each of these factors has yet to be determined.

RESERVOIR SEDIMENTATION SURVEY METHODS IN THE U.S. ARMY ENGINEER DISTRICT, VICKSBURG, MISS.

[Paper No. 81]

By OMEGA H. SHAMBLIN, *engineer, U.S. Army Engineer District, Vicksburg, Miss.*

Synopsis

The Vicksburg District has developed a practicable method[1] for performing sedimentation surveys which produces a degree of accuracy that is consistent with the conditions existing in that district, which has experienced only relatvely minor siltation. The method is also used in the periodic resurveys of these ranges.

In this development, the major departure from conventional survey methods is in the positioning of soundings along the ranges. Briefly, a taut steel piano wire is anchored along the exact alinement of each range. As the sounding boat proceeds along the range, the distances are automatically determined at 20-ft. intervals by means of a calibrated measuring wheel that is encircled by the wire, the lake bottom elevations are simultaneously recorded by a supersonic depth recorder, and alinement is maintained by radioed instructions from a transitman on the bank to the sounding boat operator.

Alinement and distance errors are considered to be within 2 ft. Plotted results of ranges profiled by this method have been compared with direct leveling profiles surveyed before the reservoir was filled. These comparisons indicate an accuracy in elevation of ±0.5 ft. in depths up to 23 ft., and ±1.0 ft. in depth up to 180 ft.

[1] This survey method was published in the January 1961 issue of *International Hydrographic Review*. This article is a revision of that publication, to reflect the changes in methods and equipment since the original article was prepared.

General

To determine the rate of silting in reservoirs, the U.S. Army Engineer District, Vicksburg, uses a system of permanently marked ranges that are usually established either during or immediately after construction of the dam. These ranges are surveyed periodically to obtain comparative profiles, which indicate the rate of silting. Figure 1 is a range layout for one of the reservoirs.

Previous Survey Methods

Prior to 1950 the permanent ranges were established and profiled by conventional direct-leveling survey methods before the reservoirs were filled. The original range layouts were to third-order accuracy. (Horizonal position closure 1/5,000, vertical closure 0.05 ft. $\sqrt{\text{length of loop in miles.}}$)

Resurveys of these ranges were made by direct leveling and taped distances on the portions of the ranges above the water surface, and by lead-line soundings below the water surface. Horizontal positions for the soundings were determined by visual signals from the sounding boat, as it traveled along each range line, and by turning angles to the boat at each sounding with a transit at a convenient location on the shore.

This method was used for a resurvey of Sardis Reservoir in northern Mississippi in 1943. The results of the underwater surveys were not considered satisfactory, because of the errors in the horizonal position of each sounding

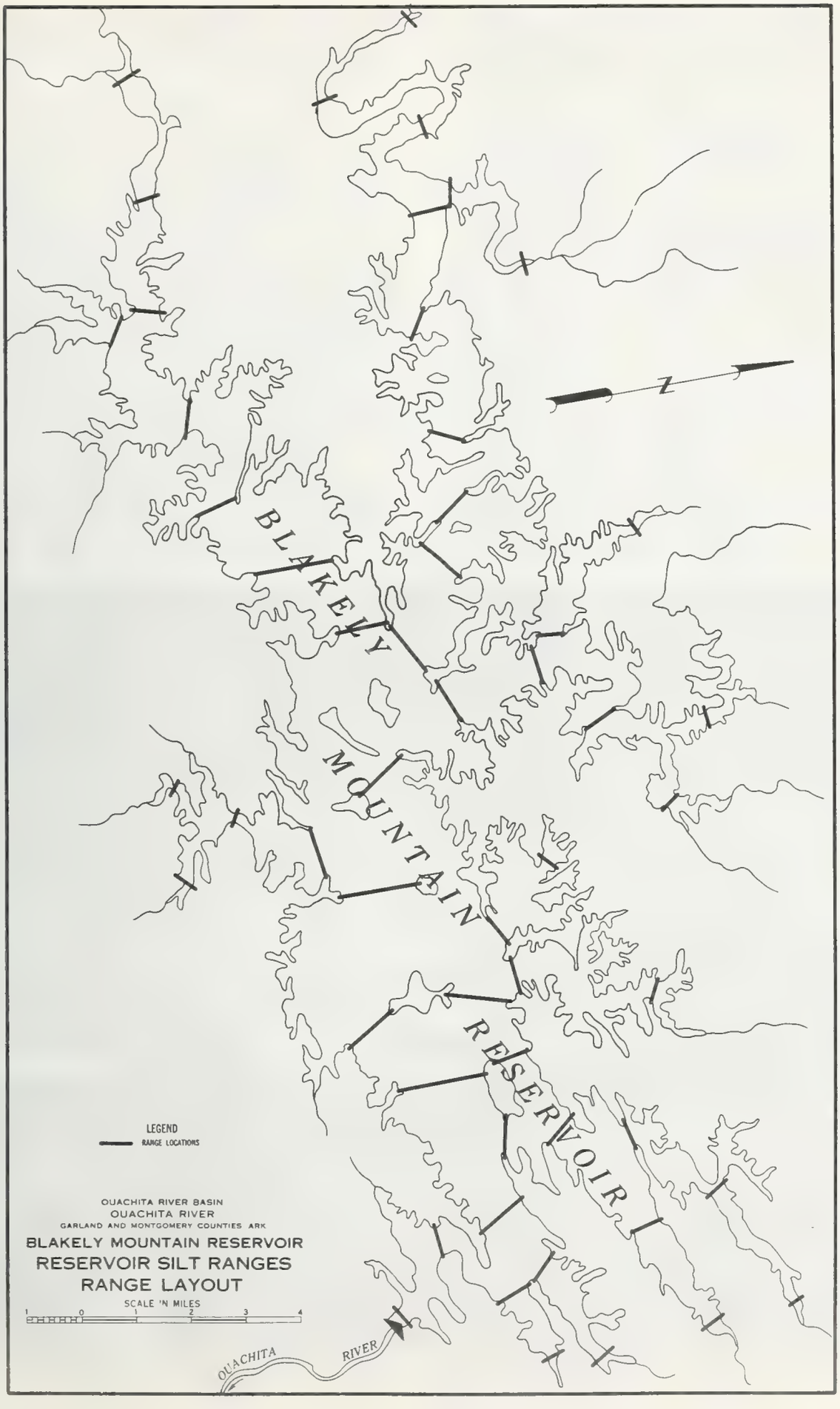


FIGURE 1.—Layout of reservoir deposition ranges of mountain reservoir.

and the inability to determine the top of the silt deposit with the sounding weight. These inaccurate results caused the Vicksburg District to search for a more reliable survey method.

Improved Method for Resurveys

During the period 1946–47, a new procedure for the underwater survey was developed and pilot models of specialized equipment were made. This improved survey method was first used during a resurvey of Arkabutla Reservoir, also in northern Mississippi, in 1947–48. The results of this survey proved that the new method would produce results that were considerably more accurate than those of the methods previously used.

In brief, the improved method consists of stretching a wire on range across the reservoir. A calibrated distance-measuring wheel traveling along this wire measures distances simultaneously as the depths are obtained with a supersonic depth recorder. Alinement directions are transmitted to the sounding-boat operator by radio from the transitman located at a point on the range as the boat proceeds along the range. Figure 2 shows the sounding boat and equipment.

Improved Method of Surveys for Establishing and Profiling Ranges

Before the construction of Blakely Mountain Dam in west-central Arkansas, it was evident that conventional surveys for establishing and profiling the permanent-range system in the rugged and mountainous terrain in which the reservoir was located would be unduly expensive. It was therefore decided to:

(1) Delay establishing the permanent ranges until after water was impounded in the reservoir;
(2) Eliminate expensive traverse in the mountains by locating the permanent-range markers by ties to aerial photographs;
(3) Use the water surface of the lake as a datum plane to eliminate the need for direct leveling for vertical control; and

FIGURE 2. — Sounding boat and equipment used in sedimentation surveys.

(4) Use the improved method developed for resurveys in making the initial underwater survey of the permanent ranges.

A summary description of the survey method follows.

Horizontal Control

Horizontal positions of the permanent-range markers are obtained by ties to aerial photographs. This eliminates traverse tielines between ranges and ties to existing control.

Vertical Control

The water surface in the reservoir is used as a datum plane for establishing elevations on the permanent-range markers. In determining the datum plane for the water surface in the Blakely Mountain Reservoir Survey, several gages were located at strategic points around the reservoir. These gages were tied to existing first- and second-order benchmarks and were read twice daily during the period of the survey. It was found that water-surface fluctuations were constant at all gages and that the differences in water-surface elevations at the various gages were never more than two-tenths of a foot. This eliminated the necessity for level tielines between ranges and ties to existing control.

Range Profiles

The portions of all ranges above the reservoir level are traversed and profiled by conventional direct-leveling survey methods. The underwater portion of each range is profiled with the special equipment and improved method described below.

Special Equipment

Special equipment needed for this operation includes supersonic depth recorder, distance wire and reel, measuring wheel, radios, and wire splicer.

Supersonic depth recorder

The depth recorder used on the Blakely Mountain Reservoir Survey was manufactured commercially.

The specifications on the outboard oscillator were:

Sound frequency.....14 kc./sec.
Beam width.....not exceeding 30°
Sounding rate.....200 per minute
Theoretical accuracy.....3 in. in 150 ft.

The depth recorder used on later surveys is a light, more easily portable instrument (see fig. 2). The specifications on the outboard oscillator are:

Sound frequency.....210 kc./sec.
Beam width.....12°
Sounding rate.....600 per minute
Theoretical accuracy.....0.5 percent

When comparing profiles obtained by depth recorder with different oscillator specifications, some difference in results can be expected. The major difference will result from the resolving capabilities of the oscillator. Other factors being equal, the oscillator with the smallest angle of beam width will be more directional and will give a more detailed definition of the surface directly beneath the oscillator.

Distance wire and reel

The distance wire used is an 0.059-in.-diameter, steel piano wire. This wire is stored on a specially built, powered reel having a drum diameter of 9 in. (fig. 3.)

Measuring wheel

The distances across the lake are measured with a specially built measuring wheel having a circumference (at the center of the wire) of 2 ft. (fig. 4). An attached counter registers the revolutions of the wheel. A cam and microswitch on the counter actuate a switch in the depth recorder to make a fix mark automatically on the depth-recorder scroll at intervals of 20 ft. The wire encircles the wheel to provide a maximum of friction and prevent slippage (fig. 5). The measuring wheel is mounted in the sounding boat directly over the depth-recorder oscillator. A light is mounted on the measuring wheel to serve as a target when, because of wind and rough water during the day, it becomes necessary to operate at night.

Radios

Transceiver radios having 0.75-w. transmitters are used for communication between the transitman, distance-reel operator, and boat operator. (fig. 6).

Wire splicer

A compressed-sleeve-type wire splicer is used for repairing breaks in the distance wire.

Method for Underwater Surveys

Stretching wire on range

A transit is set up on line at a point near one end of the range where the transitman has an unobstructed view of the range. The powered distance-wire reel is placed on range near the water's edge close to the transit. The loose end of the distance wire is then towed by boat across the lake and securely fastened at a point on range on the opposite bank. While the wire is being towed across the lake, radioed directions from the transitman guide the boat operator in keeping the boat on the range alinement. After the wire has been secured on the opposite bank, the wire is placed in an eye on a vertical post at the bow of the boat and the boat is then run back toward the transitman to place the wire on the exact range alinement. As the boat makes this return trip, the slack is taken up and

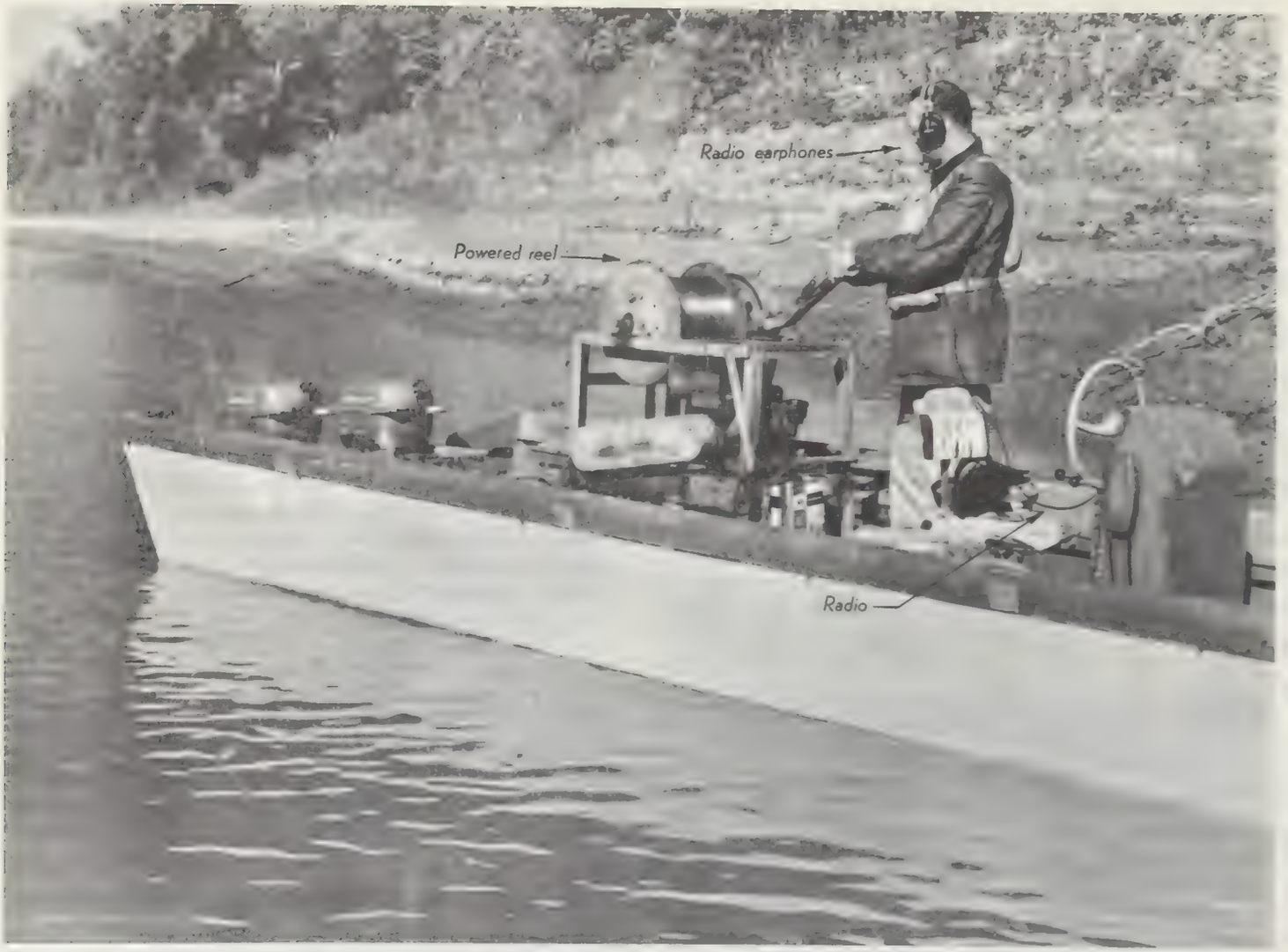


FIGURE 3. — Powered reel for piano wire.

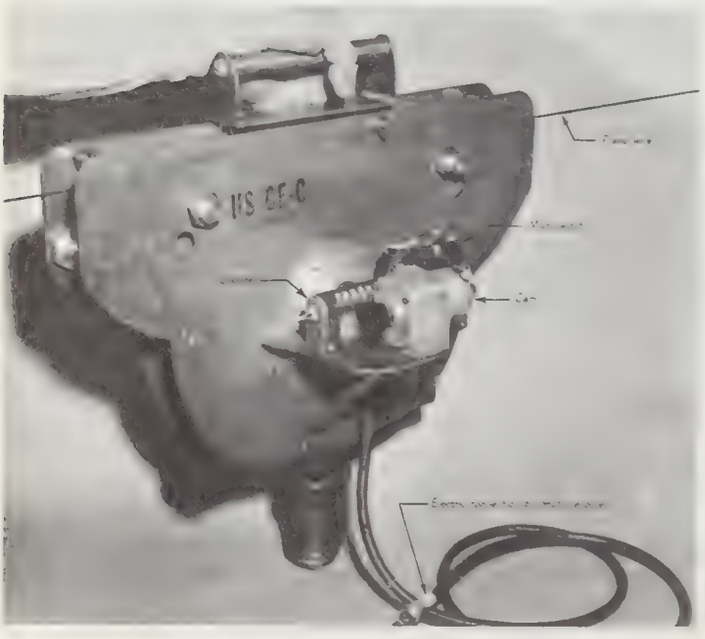


FIGURE 4. — Distance-measuring wheel.

buoys are fastened on the wire at approximate 300-ft. intervals. These floats holding up the wire reduce sag and the resulting errors in distance. Radioed alinement directions from the transitman to the boat operator are also given on this return trip. After the distance wire has been placed on range, the sounding boat is moved onto the range and the wire is threaded around the measuring wheel. The wire is then drawn taut and securely fastened at a point on the range.

Sounding the range

To establish a reference point for the sounding boat on the range stationing, the boat is securely held at a point on the distance wire and the range station at the measuring wheel is determined by a taped distance from a known station on the range (fig. 7). Before the sounding boat is moved from this point, the counter is set at zero and the cam is set under the microswitch to make the first fix mark on the

depth-recorder chart. Then as the boat moves forward on the range, the cam will strike the microswitch at 20-ft. intervals, automatically causing a fix mark to be made on the depth-recorder chart. The buoys on the wire furnish an approximate indication of alinement that enables the boat operator to evaluate the transitman's radioed directions better for maintaining a true course. On the sounding run, it is

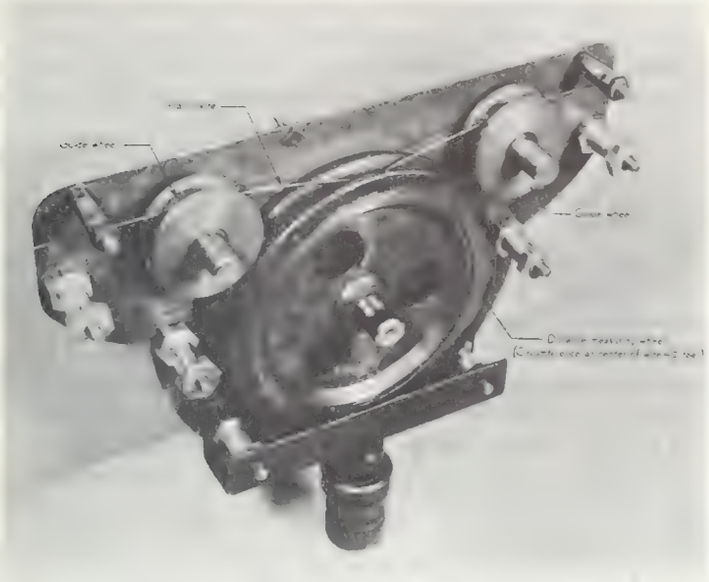


FIGURE 5. — Distance-measuring wheel with cover removed.

FIGURE 6. — Transitman at point on range.

FIGURE 7. — Sounding boat being zeroed on range.

necessary to remove and replace each buoy on the wire as the boat passes each buoy location. Figure 8 shows a sounding boat in operation on range. When the sounding boat reaches the opposite side of the reservoir, the counter reading represents the distance across the lake. The profile is extended from this point in the usual manner with levels and taped distances. Before the distance wire is recovered, the tension is released and the buoys are removed. The powered reel makes recovery of the wire more rapid and less laborious than by hand.

Results

Ranges up to 20,000 ft. have been profiled by this method. In accuracy tests on a 6,000-ft. taped course, the distances obtained by the distance-measuring wheel consistently agreed with the taped distances to the closest foot over the entire course and at numerous intermediate points. Under actual working conditions on ranges previously taped before the reservoir was filled, the measuring-wheel distances across the lake agreed with the original taped distances within ±2 ft. on distances up to 13,600 ft.

Alinement errors can be held within any reasonable limits. Under actual working conditions, an experienced boat operator can, with care, keep the oscillator within 1 or 2 ft. of the range alinement. This can be done without any appreciable loss of time.

In establishing and profiling the ranges by this method, it was considered necessary to conduct certain tests to determine if the depth-recorder survey would yield data that could be compared with direct-leveling surveys. Also, it was desirable to determine the effect of standing submerged timber in the reservoir pool upon the soundings.

To resolve these questionable points, several of the Blakely Mountain Reservoir ranges in the timbered areas were cleared to a width of 25 ft. These ranges were profiled by direct leveling before the reservoir was filled. Certain other ranges in the timbered areas were not cleared and were also profiled by direct leveling. These same ranges were resurveyed with the improved method after the reservoir was filled. Figure 9 is a chart comparing tests on one range.

A comparison of plotted results from surveys by both the direct leveling and improved methods indicates that:

(1) In Arkabutla Reservoir in depths up to 23 ft., the soundings had an accuracy of ±0.5 ft.

(2) On test ranges in Blakely Mountain Reservoir where the original survey was

FIGURE 8. — Range sounding.

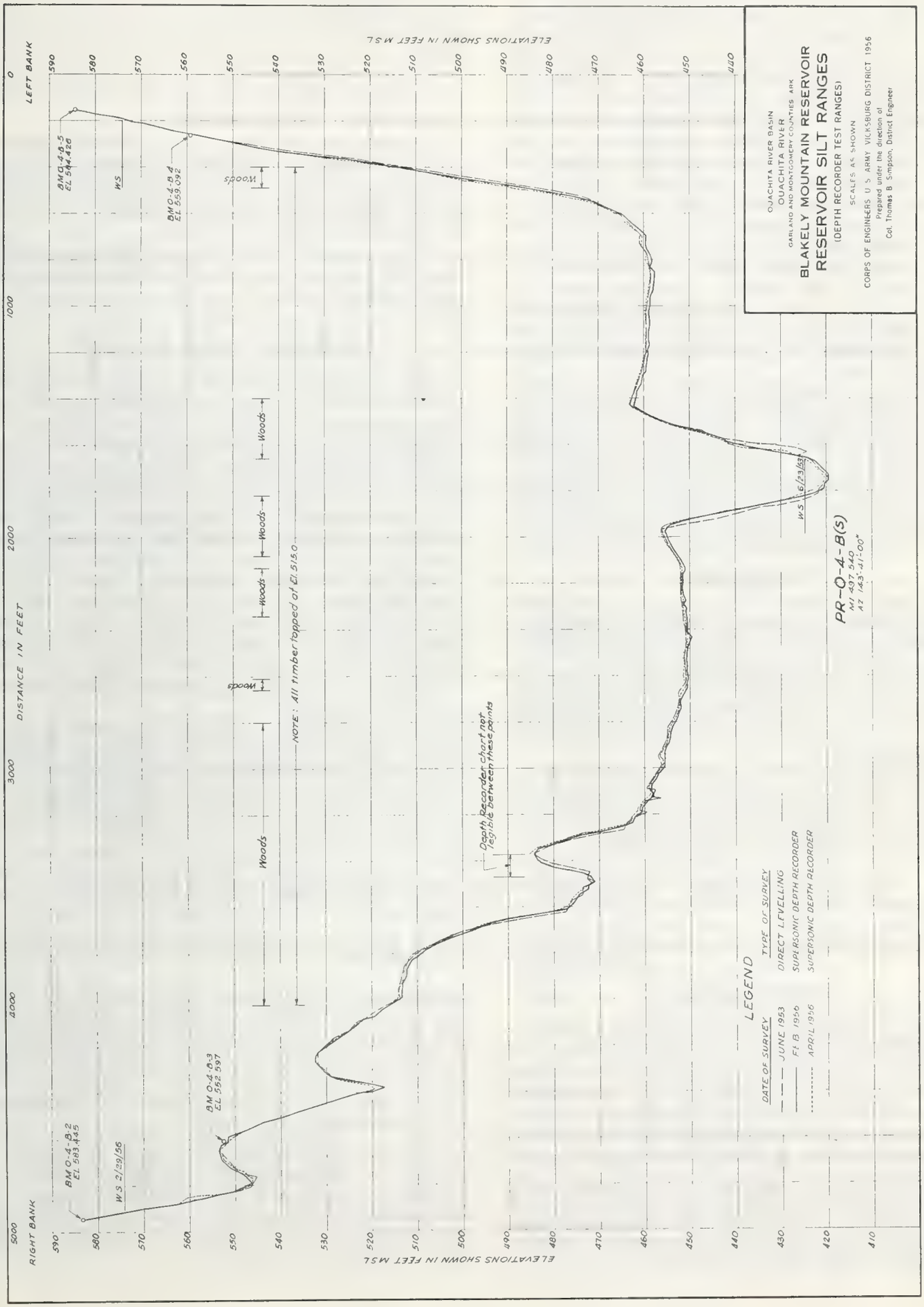


FIGURE 9.— Depth recorder test ranges, Blakeley Mountain Reservoir.

made by direct leveling and a check profile was run with the depth recorder, the soundings had a general accuracy of ±1.0 ft. in depths up to 155 ft. on the valley bottom, flat shelves, and on relatively flat slopes. However, due to the mountainous terrain where steep underwater slopes were present on or immediately adjacent to the range, the depth-recorder soundings were influenced by echoes rebounding from these steep slopes and did not give the true depth directly underneath the sounding boat. Also, the depth recorder tends to smooth out sharp irregularities on the profile in deep water. These inaccuracies are inherent in the depth finder and are controlled by the beam width of the oscillator.

(3) On test ranges in Blakely Mountain Reservoir, in check profiles obtained by the depth recorder over profiles previously surveyed by depth recorder, practically no differences were noted on the steep slopes; and the differences in the profiles on the valley bottom were ±1.0 ft. in depths up to 180 ft.

Conclusions

The establishment of the original permanent-range system on Blakely Mountain Reservoir with the improved method was completed at a considerable saving in funds.

The improved method produces results well within the accuracy of other field methods available. This is especially true in deep water and on long ranges.

In terrain where steep underwater slopes are present, office adjustment will be required if the range profiles from this method are to be compared with direct-leveling profiles.

Good results are obtained when the original profile and the resurvey are both made with the improved method at comparable water stage. In these instances, the effect of echoes from steep slopes can be ignored, because these inaccuracies are relative and are present to the same extent in both profiles.

Comparison of the depth-recorder profiles with direct-leveling profiles on ranges in cleared and wooded areas did not indicate any appreciable effect of submerged, standing timber on the depth-recorder soundings.

This improved method can be successfully used for slack-water surveys on any reasonably calm body of water.

Continuing Improvements

The Vicksburg District is considering the development of a radar positioning instrument. This equipment, if feasible, will be used on Mississippi River hydrographic surveys. However, it may be useful in sedimentation surveys, especially in reservoirs having a high rate of silting.

This equipment will automatically track and plot the position of a sounding boat traveling at random over the area to be mapped. Position "fixes" on the map and depth "fixes" on the recorder chart will be made simultaneously by remote radio facilities. Therefore, depths from the recorder chart can be added to the plotted location of the boat at each "fixed" location. Depths and positions between "fixed" points can be interpolated.

The equipment's range is expected to be 2 miles when plotting at 1/10,000 scale. The expected accuracy within this range is ±25 ft. The range of the equipment can be doubled to 4 miles with a resulting plotting scale of 1/20,000 and accuracy of ±50 ft.

INITIAL UNIT WEIGHT OF DEPOSITED SEDIMENTS

[Paper No. 82]

By J. M. LARA, *hydraulic engineer*, and E. L. PEMBERTON, *assistant head, Sedimentation Section, Hydrology Branch, Bureau of Reclamation, Denver*

Introduction

In the course of conducting project investigation studies by the Bureau of Reclamation, it is frequently necessary to determine the average unit weight of sediments expected to accumulate in a reservoir. The storage capacity of a reservoir is progressively reduced with time, depending upon the rate of the inflowing sediments. Estimating the average unit weight of these incoming sediments is necessary to determine the magnitude of the reduction in the reservoir capacity. The stream sediment transport load, generally measured in units of weight per time, can be converted to a volume quantity by means of the estimated unit weight.

It is the purpose of this paper to present a method of estimating the average initial unit weight of sediments deposited in a reservoir. The method developed is similar in principle to that of the Lane and Koelzer [1] studies and further investigated by Miller.[2]

[1] LANE, E. W., and KOELZER, V. A. DENSITY OF SEDIMENTS DEPOSITED IN RESERVOIRS. *In* A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams. St. Paul U.S. Engin. Suboffice, Hydraul. Lab., Rpt. 9, 60 pp. 1943. Iowa Univ.

Data from many sources were compiled and analyzed by means of the IBM 650 electronic computer. A mathematical procedure of the method is presented by applying it to a specific example from a given set of field data.

Nomenclature

The term, "density," has often been used by sedimentologists to describe what is more appropriately known as unit weight or specific weight. This is apparently a carryover of the term used by Lane and Koelzer in the title of their report and used as a synonym for unit weight. However, density, as defined by Rouse,[3] is expressed as *mass* per unit volume and designated as ρ (rho) ; Rouse further defines specific or unit weight as *weight* per unit volume expressed in pounds per cubic foot and has the symbol γ (gamma). In this paper the term, "unit weight," will be used.

In most sedimentation studies and investigations by the Bureau of Reclamation, the classification of sediments proposed by the American Geophysical Union is used. This classification, arranged according to type of sediments in the size ranges less than gravel, is indicated below:

Sediment type	*Size range (microns)*
Clay	Less than 4
Silt	4 to 62.5
Sand	62.5 to 2,000

Factors Affecting Unit Weight

There are several factors influencing the unit weight of sediments that deposit in a reservoir. The more dominant ones include: (1) The manner by which the reservoir is to be operated; (2) the texture and size of the sediment particles; (3) the compaction or consolidation rate of the deposited sediments; and (4) other less occurring or influential factors such as the action of density currents, thalweg slope of the incoming stream, and the effect existing vegetation in the reservoir headwaters area would have by screening out some of the incoming sediments.

Of the above-mentioned factors, probably the most influential is that of reservoir operation. Sediments depositing in a reservoir that is drawn down for considerable periods are subjected to exposure. The sediments consequently dessicate and become more dense, thus increase the unit weight. On the other hand, reservoirs normally operating with a full pool would cause the unit weights to be lower, the deposited sediments not being exposed. Reservoirs operating under fluctuating levels impose an intermediate set of conditions that produce varying effects on the unit weight of the deposits, depending on where they are located in the reservoir.

The size of the incoming sediment particles has a significant effect upon unit weights. Sediments composed of high silts and sands would have higher unit weights than those in which clay predominates.

Compaction or consolidation rates are also of importance in affecting the unit weights of deposited sediments. Koelzer and Lara[4] summarized the factors influencing these rates as (1) the weight of the overlying sediment, (2) the degree of exposure to drying, (3) particle size, (4) permeability, and (5) time. Little information is available as to the quantitative determination of the consolidation rates of the deposited sediments; therefore, they were excluded from the analysis presented in the development of the procedure in this paper.

Data

Sources of data used in the compilation (table 1) were numerous, although a major part of the data was obtained from reservoir survey reports by Federal and State agencies. A total of 1,316 samples were used, including those in the report of Lane and Koelzer.[5]

Table 1 lists information pertinent to each sample including (1) the location of the reservoir where the sample was taken, (2) observed unit weight, (3) percentages of clay, silt, and sand, (4) computed unit weight, and (5) the deviations of the computed from the observed unit weights for each sample.

The data have been arranged according to three types of reservoir operations and a fourth type that includes only river sediments. These types are described as follows:

Type	*Reservoir operation*
I	Sediment always submerged or nearly submerged
II	Normally moderate to considerable reservoir drawdown
III	Reservoir normally empty
IV	Riverbed sediments

Selecting the proper type of reservoir operation from the above tabulation to apply to a specific problem is a matter of judgment. Information from the initial reservoir operation study relative to fluctuations in the reservoir level is a valuable aid in selecting the type of reservoir

[2] MILLER, C. R. DETERMINATION OF THE UNIT WEIGHT OF SEDIMENT FOR USE IN SEDIMENT VOLUME COMPUTATIONS. Bur. of Reclamation, 7 pp. 1953.

[3] ROUSE, H. ELEMENTARY MECHANICS OF FLUIDS. 376 pp., illus. New York and London. 1946.

[4] KOELZER, V. A., and LARA, J. M. DENSITY AND COMPACTION RATES OF DEPOSITED SEDIMENT. Amer. Soc. Civil Engin. Jour. of Hydraulics Div. 84(HY2): 1603-1 to 1603-15. 1958.

[5] See footnote 1.

TABLE 1.—*Compilation of data for unit weight analysis*

TYPE I. RESERVOIR OPERATION: SEDIMENT ALWAYS SUBMERGED OR NEARLY SUBMERGED

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Harry Strunk	Nebraska	USBR	1	99.3	1.1	21.6	77.3	90.35	8.95
			2	97.1	2.4	28.8	68.8	87.75	9.35
			3	87.9	2.5	10.6	62.0	85.86	2.04
			4	88.6	4.8	23.6	71.6	87.24	1.36
			5	80.7	8.2	77.0	14.8	70.53	10.17
			6	94.3	5.1	59.0	35.9	77.53	16.78
			7	72.1	10.2	86.5	3.3	66.41	5.69
			8	85.7	4.5	78.4	17.1	72.83	12.87
			9	84.3	4.6	73.9	21.5	73.24	10.53
			10	84.3	6.3	79.3	14.4	71.74	13.16
			11	91.4	4.5	68.5	27.0	75.53	15.87
			12	74.2	9.6	80.4	10.0	68.30	5.90
			13	79.6	3.9	77.2	18.9	73.37	6.23
			14	52.1	27.4	68.0	4.6	59.47	−7.37
			15	78.6	11.5	76.9	11.6	67.96	10.64
			16	61.4	9.8	71.4	18.8	70.73	−9.33
			17	45.7	36.4	61.0	2.6	54.97	−9.27
			18	39.9	16.2	75.6	8.2	65.12	−25.22
			19	30.0	60.7	38.0	1.3	43.43	−13.43
			20	47.7	25.9	70.6	3.5	59.64	−11.94
			21	60.4	11.8	83.6	4.6	66.07	−5.67
			22	36.5	49.2	49.9	.9	48.71	−12.21
Leavenworth County State Lake.	Kansas	SCS	23	58.0	25.6	72.4	2.0	59.10	−1.10
			24	72.0	23.2	68.8	8.0	62.04	9.96
			25	57.0	37.6	61.4	1.0	53.55	3.45
			26	73.0	20.0	77.0	3.0	62.01	10.99
			27	55.0	52.8	46.2	1.0	46.95	8.05
			28	70.0	39.2	53.8	7.0	54.73	15.27
			29	37.0	55.0	44.0	1.0	46.07	−9.07
			30	38.0	24.0	74.0	2.0	59.98	−21.98
			31	59.0	39.2	57.8	3.0	53.65	5.35
Lake Calhoun	Illinois	SCS	32	47.5	39.4	60.5	.1	52.84	−5.34
			33	68.0	24.9	74.7	.4	59.00	9.00
Lake Bracken	Illinois	SCS	34	48.5	38.9	60.0	1.1	53.11	−4.61
			35	39.4	41.4	58.0	.6	52.23	−12.83
			36	36.5	45.4	53.9	.7	50.47	−13.97
Lancaster Reservoir	South Carolina	SCS	37	53.6	61.4	37.6	1.0	43.43	10.17
			38	70.0	36.2	59.1	4.7	55.51	14.49
			39	70.2	20.0	67.0	13.0	64.71	5.49
			40	78.9	13.3	70.0	16.7	68.87	10.03
			41	68.4	24.8	68.2	7.0	60.89	7.51
			42	39.4	25.1	65.5	9.4	61.43	−22.03
			43	61.5	20.6	71.4	8.0	62.92	−1.42
Conchas Reservoir	New Mexico	CE	44	20.7	67.0	32.0	1.0	40.79	−20.09
			45	84.0	4.0	14.0	82.0	90.38	−6.38
			46	64.2	19.0	80.0	1.0	61.91	2.29
			47	45.7	20.0	73.0	7.0	63.09	−17.39
			48	28.0	63.0	37.0		42.28	−14.28
			49	59.2	53.0	44.0	3.0	47.49	11.71
			50	40.5	40.0	55.0	5.0	53.75	−13.25
			51	35.1	51.0	43.0	6.0	49.18	−14.08
Tarryall Reservoir	Colorado	FS	52	87.4	1.6	8.6	89.8	93.42	−6.02
			53	31.2	19.2	72.6	8.2	63.80	−32.60
			54	81.1	4.8	16.8	78.4	88.86	−7.76
Lake McBride	Iowa	SCS	55	59.0	22.0	77.0	1.0	60.59	−1.59
			56	59.0	20.0	77.0	3.0	62.01	−3.01
			57	62.0	17.0	81.0	2.0	63.06	−1.06
			58	56.0	16.0	80.0	4.0	64.04	−8.04
			59	60.0	14.0	85.0	1.0	64.11	−4.11
Lake Olathe	Kansas	SCS	60	60.0	22.4	22.1	54.5	74.90	−14.90
Lake El Dorado	Kansas	SCS	61	66.0	15.2	27.1	57.5	78.25	−12.25
Fort Supply Reservoir	Oklahoma	CE	62	41.3	56.0	39.0	5.0	46.71	−5.41
			63	35.5	72.0	24.0	4.0	39.40	−3.90
			64	33.5	77.0	20.0	3.0	36.93	−3.43
			65	30.5	79.0	19.0	2.0	35.78	−5.28
			66	63.9	32.0	26.0	42.0	67.26	−3.36
			67	37.5	68.0	30.0	2.0	40.62	−3.12

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Fort Supply Reservoir Continued	Oklahoma	CE	68	33.5	74.0	25.0	1.0	37.71	−4.21
			69	38.6	64.0	25.0	1.0	37.71	.89
			70	38.5	78.0	30.0	2.0	40.62	−2.12
			71	34.9	69.0	28.0	3.0	40.45	−5.55
			72	55.1	62.0	35.0	3.0	43.53	11.57
			73	37.5	75.0	22.0	3.0	37.81	−.31
			74	58.5	53.0	36.0	11.0	49.65	8.85
			75	35.1	78.0	21.0	1.0	35.95	−.85
			76	38.2	72.0	22.0	6.0	39.94	−1.74
			77	101.0	9.0	17.0	74.0	86.02	14.98
			78	58.2	51.0	19.0	30.0	55.66	2.45
			79	119.9	2.0	2.0	96.0	95.04	24.86
			80	73.8	47.0	26.0	27.0	56.61	17.19
			81	60.9	58.0	38.0	4.0	45.56	15.34
			82	117.5	3.0	1.0	96.0	94.60	22.90
			83	97.0	7.0	14.0	79.0	88.25	8.75
			84	80.9	28.0	38.0	34.0	66.86	14.04
			85	77.5	25.0	39.0	36.0	68.72	8.78
			86	98.4	2.0	3.0	95.0	94.77	3.63
			87	111.0	3.0	4.0	93.0	93.79	17.21
Lake Texoma	Okla. and Texas	CE	88	41.0	62.0	37.0	1.0	42.99	−1.99
			89	44.0	52.0	46.0	2.0	47.66	−3.66
			90	48.0	41.0	56.0	3.0	52.77	−4.77
			91	64.0	44.0	48.0	8.0	52.80	11.20
			92	50.0	34.0	48.0	23.0	61.25	−11.25
			93	57.0	36.0	52.0	12.0	57.40	−.40
			94	87.0	20.0	72.0	8.0	63.36	23.64
			95	69.0	30.0	47.0	23.0	63.01	5.99
			96	40.0	66.0	26.0	8.0	43.12	−3.12
			97	86.0	15.0	64.0	21.0	69.07	16.93
			98	56.0	42.0	49.0	9.0	53.95	2.05
			99	71.0	55.0	43.0	2.0	46.34	24.66
			100	76.0	13.0	67.0	20.0	69.68	6.32
			101	86.0	50.0	49.0	1.0	48.27	37.73
			102	74.0	20.0	75.0	5.0	62.55	11.45
			103	90.0	8.0	65.0	27.0	73.77	16.23
			104	83.0	9.0	66.0	25.0	72.79	10.21
			105	85.0	0	22.0	78.0	91.06	−6.06
			106	53.0	30.0	67.0	3.0	57.61	−4.61
			107	87.0	7.0	26.0	67.0	85.01	1.99
			108	79.0	28.0	64.0	8.0	59.84	19.16
			109	89.0	11.0	59.0	30.0	73.26	15.74
			110	74.0	12.0	60.0	28.0	72.28	1.72
			111	53.0	11.0	55.0	34.0	74.34	−21.34
			112	86.0	6.0	41.0	53.0	81.67	4.33
			113	89.0	8.0	61.0	31.0	74.85	14.15
			114	99.0	3.0	25.0	72.0	88.12	10.88
			115	98.0	3.0	3.0	94.0	94.06	3.94
			116	87.0	5.0	19.0	76.0	88.32	−1.32
			117	63.0	35.0	39.0	26.0	61.62	1.38
			118	53.0	36.0	49.0	15.0	58.21	−5.21
			119	35.0	73.0	27.0	0	37.88	−2.88
			120	86.0	11.0	36.0	53.0	79.47	6.53
			121	49.0	37.0	55.0	8.0	55.88	−6.88
			122	88.0	16.0	37.0	47.0	75.65	12.35
			123	39.0	74.0	25.0	1.0	37.71	1.29
			124	52.0	53.0	41.0	6.0	48.30	3.70
			125	49.0	56.0	47.0	7.0	47.25	1.75
			126	88.0	50.0	47.0	3.0	48.81	39.19
			127	88.0	12.0	64.0	24.0	71.20	16.80
			128	48.0	59.0	37.0	4.0	45.12	2.88
			129	82.0	30.0	46.0	24.0	63.28	18.72
			130	93.0	6.0	59.0	35.0	76.81	16.19
			131	58.0	45.0	35.0	20.0	55.60	2.40
			132	92.0	9.0	68.0	23.0	72.25	19.75
			133	44.0	60.0	28.0	2.0	44.14	−0.14
			134	91.0	26.0	71.0	3.0	59.37	31.63
			135	98.0	3.0	13.0	84.0	91.36	5.64

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Lake Texoma—Continued	Okla. and Texas...	CE.....	136	79.0	20.0	25.0	55.0	76.05	2.95
			137	100.0	4.0	26.0	70.0	87.14	12.86
			138	89.0	3.0	37.0	60.0	84.88	4.12
			139	91.0	5.0	25.0	70.0	86.70	4.30
			140	97.0	3.0	22.0	57.0	88.93	8.07
			141	89.0	6.0	37.0	57.0	82.75	6.25
			142	81.0	7.0	43.0	50.0	80.42	0.58
			143	87.0	3.0	17.0	80.0	90.28	−3.28
			144	96.0	3.0	1.0	96.0	94.70	1.40
			145	96.0	2.0	3.0	95.0	94.77	1.23
			146	108.0	3.0	1.0	96.0	94.60	13.40
Lake Bloomington....	Illinois.....	Ill. State Water Survey.	147	74.26	60.0	23.0	17.0	48.19	26.07
			148	84.24	74.0	15.0	11.0	40.41	43.83
			149	41.81	86.0	13.0	1.0	32.43	9.38
			150	42.43	87.0	12.0	1.0	31.99	10.44
			151	42.43	88.0	11.0	1.0	31.55	10.88
			152	36.19	89.0	9.0	2.0	31.38	4.81
			153	41.18	81.0	12.0	7.0	36.25	4.93
			154	27.46	86.0	11.0	3.0	32.97	−5.51
			155	28.08	92.0	7.0	1.0	29.79	−1.71
			156	29.33	86.0	11.0	3.0	32.97	−3.64
			157	32.45	91.0	6.0	3.0	30.77	1.68
			158	64.90	65.0	17.0	18.0	46.26	18.64
			159	56.78	77.0	20.0	3.0	36.93	19.85
			160	45.55	77.0	22.0	1.0	36.39	9.16
			161	49.30	81.0	18.0	1.0	34.63	14.67
			162	41.81	85.0	14.0	1.0	32.87	8.94
Lake Decatur.....	Illinois.....	SCS.....	163	39.94	43.4	38.6	18.0	55.94	−16.00
			164	34.32	66.2	32.3	1.5	41.50	−7.18
			165	24.96	67.8	32.0	.2	40.08	−15.12
			166	24.96	35.5	58.9	5.6	55.78	−30.82
			167	37.44	57.7	39.4	2.9	45.29	−7.85
			168	37.44	56.5	42.7	.8	45.63	−8.19
			169	36.82	65.0	34.6	.4	41.40	−4.58
			170	24.34	56.2	43.2	.6	45.63	−21.29
			171	19.97	57.8	41.4	.8	44.75	−24.78
			172	33.07	63.3	36.5	.2	42.28	−9.21
			173	33.07	46.8	52.8	.4	49.32	−16.25
			174	33.07	66.7	33.0	.3	40.52	−7.45
			175	30.58	73.4	26.4	.2	37.88	−7.30
			176	29.33	76.9	23.0	.1	36.12	−6.79
			177	21.84	58.3	40.9	.8	44.75	−22.91
			178	36.82	54.1	45.7	.2	46.24	−9.42
			179	39.31	55.3	44.5	.2	45.80	−6.49
			180	42.43	56.9	43.1	.0	44.92	−2.49
			181	38.69	51.3	48.6	.1	47.56	−8.87
			182	35.57	54.7	41.4	3.9	46.88	−11.31
			183	38.69	62.2	37.0	.8	42.99	−4.30
			184	41.81	59.3	38.7	2.0	44.58	−2.77
			185	38.06	54.7	44.5	.8	46.07	−8.01
			186	34.94	64.4	34.9	.7	42.11	−7.17
			187	38.06	54.3	45.5	.2	46.24	−8.18
			188	42.43	61.4	38.5	.1	43.16	−0.73
			189	35.57	57.8	42.1	.1	44.48	−8.91
			190	41.81	51.8	48.1	.1	47.12	−5.31
			191	33.07	50.7	44.1	.2	47.56	−14.49
			192	31.20	47.8	52.1	.1	48.88	−17.68
			193	37.44	52.3	47.5	.2	47.12	−9.68
			194	41.81	43.6	56.2	.2	50.64	−8.83
			195	48.67	47.4	52.5	.1	49.32	−.65
			196	48.67	43.8	56.1	.1	50.64	−1.97
			197	51.79	47.1	52.8	.1	49.32	2.47
			198	49.30	48.0	51.7	.3	48.88	−.42
			199	53.04	47.2	52.7	.1	49.32	3.72
			200	49.92	45.3	54.5	.2	50.20	−.28
			201	56.78	45.5	54.0	.5	50.20	6.58
			202	70.51	29.1	67.4	3.5	58.32	12.19
			203	70.51	11.8	20.7	67.5	82.81	−12.30

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*L/cu. ft.*	
Lake Decatur—Continued	Illinois	SCS	204	68.02	32.4	66.9	.7	56.19	11.83
			205	64.27	30.1	69.4	.5	56.80	7.47
			206	56.16	36.9	56.1	7.0	55.61	.55
			207	62.40	33.0	65.5	1.5	55.75	6.65
			208	31.82	66.1	33.7	.2	40.96	−9.14
			209	36.82	59.5	40.0	.5	44.04	−7.22
			210	43.06	33.4	62.7	3.9	56.56	−13.50
			211	44.30	47.3	52.3	.4	49.32	−5.02
			212	31.20	67.2	32.3	.5	40.52	−9.32
			213	34.32	55.8	43.9	.3	45.36	−11.04
			214	36.82	61.1	38.6	.3	43.16	−6.34
			215	38.69	41.2	58.7	.1	51.96	−13.27
			216	63.65	30.4	66.2	3.4	57.61	6.04
			217	44.93	42.3	55.5	2.2	52.06	−7.13
			218	51.79	43.1	56.7	.2	51.08	.71
			219	43.68	39.8	59.6	.6	52.67	−8.99
			220	49.92	42.9	56.7	.4	51.08	−1.16
La Von Reservoir	Texas	CE	221	54.3	43.0	48.2	8.8	53.51	.79
			222	36.2	69.0	27.9	3.1	40.45	−4.25
			223	29.4	85.5	14.3	.2	32.60	−3.20
			224	46.9	67.0	32.2	.8	40.79	6.12
			225	55.6	55.2	42.9	1.9	46.34	9.26
			226	31.5	80.0	19.1	.9	35.07	−3.57
			227	34.7	83.0	16.7	.3	33.48	1.22
			228	45.2	63.0	35.7	1.3	42.55	2.65
			229	39.2	69.5	29.2	1.3	39.91	−.71
			230	46.9	54.0	33.8	12.2	49.48	−2.58
			231	69.1	70.6	28.5	.9	39.03	30.07
			232	60.0	61.0	37.8	1.2	43.43	16.57
			233	25.4	78.4	19.8	1.8	36.22	−11.72
			234	30.9	70.8	23.0	6.2	40.38	−9.48
			235	36.9	61.2	32.7	6.1	44.78	−7.88
			236	22.0	73.8	24.1	2.1	37.98	−15.98
			237	65.2	53.5	45.4	1.1	46.95	18.25
			238	52.4	53.8	44.6	1.6	46.78	5.62
			239	69.6	47.0	37.2	15.8	53.64	15.96
			240	73.5	40.0	45.5	14.5	56.18	17.32
			241	57.4	41.0	48.2	10.8	54.93	2.47
			242	36.9	65.5	33.1	1.4	41.67	−4.77
			243	37.0	77.8	21.7	.5	35.68	1.32
			244	56.5	47.5	44.9	7.6	51.48	4.92
			245	72.0	69.4	29.7	.9	39.91	32.09
			246	70.5	63.5	33.1	3.4	43.09	27.41
			247	60.0	75.0	24.1	.9	37.27	22.73
			248	72.0	70.0	29.1	.9	29.47	32.53
			249	60.5	63.8	29.5	6.7	43.73	16.77
Hulah Reservoir	Oklahoma	CE	250	50.7	48.0	40.0	12.0	52.12	−1.42
			251	37.6	62.0	37.0	1.0	42.99	−5.39
			252	52.8	38.0	59.0	3.0	53.28	−.48
			253	42.1	63.0	34.0	3.0	43.09	−.99
			254	40.7	52.0	45.0	3.0	47.93	−7.23
			255	41.5	58.0	37.0	5.0	45.83	−4.33
			256	40.4	56.0	40.0	4.0	46.44	−6.04
			257	36.5	72.0	25.0	3.0	39.13	−2.63
			258	40.7	64.0	35.0	1.0	42.11	−1.41
			259	86.3	25.0	26.0	49.0	72.23	14.07
			260	67.2	44.0	26.0	30.0	58.74	8.46
			261	63.0	29.0	61.0	10.0	59.94	3.06
			262	62.6	32.0	39.0	29.0	75.80	−13.20
TYPE II.—RESERVOIR OPERATION: NORMALLY A MODERATE TO CONSIDERABLE RESERVOIR DRAWDOWN									
Berlin Reservoir	Pittsburg District	CE	1	63.4	60.0	38.0	2.0	49.92	13.48
			2	73.4	50.0	44.0	6.0	54.56	18.84
			3	78.9	34.0	61.0	5.0	60.06	18.84
			4	69.0	19.0	43.0	38.0	74.04	−5.04
			5	43.3	21.0	52.0	27.0	70.46	−27.16
			6	82.8	25.0	69.0	6.0	63.56	19.24

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Berlin Reservoir—Continued	Pittsburg District.	CE	7	76.4	2.0	7.0	91.0	93.94	−17.54
			8	48.5	12.0	33.0	55.0	80.98	−32.48
			9	71.0	47.0	39.0	14.0	57.72	13.28
Seminoe Reservoir	Wyoming	USBR	10	45.6	61.0	38.9	.1	49.04	−3.44
			11	34.8	55.0	43.5	1.5	51.59	−16.79
			12	44.8	54.5	41.0	4.5	52.55	−7.75
Elephant Butte Reservoir.	New Mexico	USBR	13	68.1	20.8	51.8	27.4	70.46	−2.36
			14	87.8	10.4	39.2	50.4	80.40	7.40
			15	85.7	4.0	34.3	61.7	85.68	.02
			16	76.0	44.0	47.6	8.4	57.24	18.76
			17	75.8	43.2	47.4	9.4	57.86	17.94
			18	69.9	70.4	27.6	2.0	46.32	23.58
			19	66.6	57.6	36.9	5.5	51.69	14.91
			20	90.0	19.2	15.8	65.0	81.06	8.94
			21	66.8	60.8	33.5	5.7	50.60	16.20
			22	80.7	22.4	52.2	25.4	69.58	11.12
			23	88.5	9.6	69.0	21.4	72.86	15.64
			24	93.8	8.0	65.3	26.7	75.14	18.66
			25	76.9	45.6	45.6	8.8	56.78	20.12
			26	76.7	26.4	61.6	12.0	64.76	11.94
			27	66.2	71.2	26.1	2.7	46.22	19.98
			28	62.4	72.8	24.0	3.2	45.50	16.90
			29	81.6	17.6	31.9	50.5	77.79	3.81
			30	64.8	68.8	27.2	4.0	47.20	17.60
			31	58.0	65.6	29.7	4.7	48.54	9.46
			32	73.5	31.2	14.5	54.3	73.88	.38
			33	87.7	17.6	25.2	57.2	79.34	8.36
			34	82.5	29.6	30.7	39.7	70.60	11.90
			35	85.0	10.4	60.9	28.7	74.94	10.06
			36	88.8	6.4	17.4	76.2	68.60	.20
			37	86.4	5.6	32.2	62.2	84.96	1.44
			38	72.6	36.0	43.1	20.9	63.50	9.10
			39	66.7	43.2	47.4	9.4	57.86	8.84
			40	71.1	32.0	55.5	12.5	62.73	8.37
			41	52.7	48.0	44.5	7.5	55.67	−2.97
			42	82.1	12.0	35.5	52.5	80.33	1.77
			43	35.7	64.0	31.8	4.2	49.00	−13.30
			44	35.3	60.8	34.4	4.8	50.34	−15.04
			45	28.5	72.0	26.3	1.7	45.60	−17.10
			46	30.9	73.6	25.1	1.3	44.62	−13.72
			47	76.8	13.5	50.1	36.4	75.60	1.20
			48	100.0	1.6	6.2	92.2	94.20	5.80
			49	79.3	18.4	50.4	31.2	72.58	6.72
			50	66.2	21.6	57.0	21.4	68.54	−2.34
			51	40.9	68.0	29.5	2.5	47.17	−6.27
			52	39.0	75.2	23.8	1.0	44.26	−5.26
			53	16.3	66.4	30.8	2.8	48.02	−31.72
			54	29.9	69.6	28.2	2.2	46.32	16.42
			55	40.9	72.0	26.3	1.7	45.60	−4.70
			56	34.4	71.2	27.0	1.8	45.96	−11.56
			57	31.8	52.0	42.2	5.8	53.84	−22.04
			58	35.5	63.2	33.3	3.5	49.16	−13.66
			59	40.1	72.8	25.7	1.5	45.18	−5.08
			60	28.3	58.4	37.1	4.5	51.15	−22.85
			61	34.7	71.2	27.0	1.8	45.96	−11.26
			62	40.2	68.0	29.5	2.5	47.17	−6.97
			63	38.8	74.4	24.4	1.2	44.62	−5.82
			64	32.9	68.8	28.9	2.3	46.68	−13.78
			65	39.7	71.2	27.0	1.8	45.96	−6.26
			66	24.8	68.0	29.5	2.5	47.17	−22.37
			67	31.6	76.8	22.5	.7	43.54	−11.94
			68	39.0	73.6	25.1	1.3	44.62	−5.62
			69	19.6	65.6	31.4	3.0	48.02	−28.42
			70	29.3	75.2	23.8	1.0	44.26	−14.96
			71	39.6	73.6	25.1	1.3	44.62	−5.02
			72	19.6	69.6	28.2	2.2	46.32	−26.72
			73	29.1	75.2	23.8	1.0	44.26	−15.16
			74	38.2	72.0	26.3	1.7	45.60	−7.40

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Elephant Butte—Continued	New Mexico	USBR	75	18.8	69.6	28.2	2.2	46.32	−27.52
			76	29.9	72.8	25.7	1.5	45.18	−15.28
			77	36.5	74.4	24.4	1.2	44.62	−8.12
			78	30.4	70.4	27.6	2.0	46.32	−15.92
			79	36.1	61.6	34.6	3.8	49.72	−13.62
			80	41.0	72.0	26.3	1.7	45.60	−4.60
			81	25.8	72.8	25.7	1.5	45.18	−19.38
			82	34.3	73.6	25.1	1.3	44.62	−10.32
			83	41.0	73.6	25.1	1.3	44.62	−3.62
			84	29.6	76.0	23.2	.8	43.90	−14.30
			85	35.9	58.4	33.8	7.8	52.20	−16.30
			86	42.7	70.4	27.6	2.0	46.32	−3.62
			87	39.0	69.6	23.2	2.2	46.32	−7.32
			88	67.28	48.5	51.3	.2	53.72	13.56
			89	76.63	17.0	47.2	35.8	74.24	2.39
			90	64.17	32.8	66.7	.5	59.32	4.85
			91	72.27	17.5	80.3	2.2	65.27	7.00
			92	64.17	77.0	21.4	1.6	43.80	20.37
			93	71.65	74.5	25.3	.2	44.23	27.42
			94	70.40	82.5	17.1	.4	41.40	29.00
			95	82.86	11.6	59.7	28.7	74.22	8.64
			96	74.14	84.3	15.5	.2	40.76	33.38
			97	58.56	75.1	24.5	.4	44.00	14.56
			98	72.27	34.7	35.6	29.7	66.20	6.07
			99	79.12	20.7	65.9	13.4	66.82	12.30
			100	77.25	67.2	32.3	.5	46.94	30.31
			101	87.84	5.4	51.4	43.2	80.38	7.46
			102	87.84	3.1	23.1	73.8	89.16	−1.32
			103	74.14	18.1	73.3	8.6	66.86	7.28
			104	85.97	14.1	18.8	67.1	83.38	2.59
			105	37.90	58.0	40.0	2.0	50.64	−12.74
			106	37.46	71.0	27.5	1.5	45.83	−8.37
			107	35.69	82.0	18.0	0	41.48	−5.79
			108	34.07	76.0	24.0	0	43.64	−9.57
			109	33.44	83.0	17.0	0	41.12	−7.68
			110	29.50	69.0	29.0	2.0	46.68	−17.18
			111	33.96	70.0	29.0	1.0	46.06	−12.10
			112	36.23	83.0	17.0	0	41.12	−4.89
			113	33.03	85.0	14.0	1.0	40.66	−7.63
			114	34.07	78.0	21.0	1.0	43.18	−9.11
			115	31.76	81.0	19.0	0	41.84	−10.08
			116	36.54	76.0	23.0	1.0	43.90	−7.36
			117	35.39	81.0	19.0	0	41.84	−6.45
			118	42.50	65.0	35.0	0	47.60	−5.10
			119	33.67	84.0	16.0	0	40.76	−7.09
			120	38.62	77.0	22.0	1.0	43.54	−4.92
			121	54.80	75.0	25.0	0	44.00	10.80
			122	65.41	79.0	21.0	0	42.56	22.85
			123	67.48	66.0	32.0	2.0	47.76	19.72
			124	77.40	21.0	43.0	36.0	72.80	4.60
			125	38.40	79.0	21.0	0	42.56	−4.16
			126	30.24	80.0	19.0	1.0	42.46	−12.22
Lake Carthage	Illinois	Ill. State Water Survey.	127	60.5	23.0	76.0	1.0	62.98	−2.48
			128	68.0	20.2	78.4	1.4	64.06	3.94
			129	46.8	30.9	69.0	.1	59.84	−13.40
			130	46.2	35.2	64.7	.1	58.40	−12.20
			131	45.6	35.8	63.8	.4	58.04	−12.44
			132	40.6	43.2	56.7	.1	55.52	−14.92
			133	44.3	40.5	59.4	.1	56.45	−12.15
			134	41.2	47.8	52.1	.1	53.72	−12.52
			135	46.8	39.9	59.8	.3	56.60	−9.80
			136	41.8	24.2	75.7	.1	62.36	−20.56
			137	60.3	21.4	75.2	3.4	64.22	−3.92
Altus Reservoir	Oklahoma	USBR	138	26.3	19.3	66.2	14.5	67.82	−41.52
			139	26.7	38.7	59.1	2.2	57.48	−30.78
			140	53.6	24.2	50.9	24.9	68.86	−15.26
			141	36.5	32.8	57.0	10.2	61.72	−25.22
			142	91.6	18.3	21.9	59.8	80.12	11.48

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Altus Reservoir—Continued	Oklahoma	USBR	143	106.6	1.1	10.2	88.7	93.78	12.82
			144	45.8	36.8	62.4	.8	57.94	−12.14
			145	65.4	4.6	45.3	50.1	82.20	−16.80
			146	80.8	21.1	44.4	34.5	72.37	8.43
			147	54.3	49.2	48.5	2.3	53.88	.42
			148	57.9	20.2	58.0	21.8	69.52	−11.62
			149	32.8	31.6	53.1	15.3	63.38	−30.58
			150	39.6	47.1	51.7	1.2	54.34	−14.74
			151	43.1	46.9	51.1	2.0	54.60	−11.50
			152	49.3	48.9	50.3	.8	53.62	−4.32
			153	36.1	44.0	52.9	3.1	55.94	−19.84
			154	48.6	18.4	50.1	31.5	72.57	−23.97
			155	51.9	16.0	64.7	19.3	70.18	−18.28
			156	56.3	27.0	44.7	28.3	68.56	−12.26
			157	38.3	18.0	69.9	12.1	67.64	−29.34
			158	49.8	21.0	36.4	42.6	74.62	−24.82
			159	19.9	53.0	42.6	4.4	52.96	−33.06
			160	19.3	47.4	46.0	6.6	55.90	−36.60
			161	79.8	5.8	48.7	45.5	80.74	−.94
			162	62.2	36.5	24.6	38.9	67.97	5.77
			163	82.6	9.3	61.7	29.0	75.30	7.30
			164	63.2	16.6	15.3	68.1	82.56	−19.36
			165	88.1	15.0	29.8	55.2	79.90	8.20
			166	63.5	53.0	40.5	6.5	53.61	9.89
			167	61.5	59.0	35.4	5.6	51.32	10.18
			168	101.7	3.8	15.3	80.9	90.62	11.08
			169	69.9	36.4	50.8	12.8	61.42	8.48
			170	78.4	32.0	35.6	32.4	67.80	10.60
			171	82.3	50.8	37.1	12.1	55.76	26.54
			172	97.8	12.8	14.0	73.2	85.30	12.50
			173	95.1	0	2.8	97.2	96.22	−1.12
			174	82.3	13.0	35.2	51.8	79.84	2.46
			175	88.5	3.8	19.9	76.3	89.32	−.82
			176	88.5	5.7	19.2	75.1	88.34	.16
Lake Brownwood	Texas	Water Improvement District No. 1.	177	39.6	60.8	25.2	14.0	52.68	−13.08
			178	32.7	57.6	35.2	7.2	51.94	−19.24
			179	22.2	67.2	23.5	9.3	49.22	−27.02
			180	42.0	66.4	24.9	8.7	49.58	−7.58
			181	33.8	57.6	29.4	13.0	53.50	−19.70
			182	39.8	66.4	24.1	9.5	49.57	−9.77
			183	42.9	59.2	32.3	8.5	51.90	−9.00
			184	41.1	62.4	28.9	8.7	51.02	−9.92
			185	49.3	56.0	32.3	11.7	53.96	−4.66
			186	50.2	48.0	37.0	15.0	57.62	−7.42
			187	32.7	71.2	21.1	7.7	47.52	−14.82
			188	36.4	66.4	24.9	8.7	49.58	−13.18
			189	38.4	68.8	27.2	4.0	47.20	−8.80
			190	43.8	62.4	25.6	12.0	51.80	−8.00
			191	40.2	67.2	24.3	8.5	49.02	−8.82
			192	35.6	72.0	24.7	3.3	45.86	−10.26
			193	33.9	67.2	23.5	9.3	49.22	−10.32
			194	42.2	59.2	30.6	10.2	52.36	−10.16
			195	50.9	48.0	37.0	15.0	57.62	−6.72
			196	48.4	45.6	39.7	14.7	58.34	−9.94
			197	45.6	56.0	30.7	13.3	54.22	−8.62
			198	33.6	68.8	20.2	11.0	49.02	−15.42
			199	51.1	45.6	33.9	20.5	59.91	−8.81
Angostura Reservoir	South Dakota	USBR	200	41.9	55.9	44.1	0	50.84	−8.94
			201	38.5	56.5	43.5	0	50.66	−12.16
			202	45.3	53.9	46.1	0	51.56	−6.26
			203	85.6	8.9	33.4	57.7	82.84	2.76
			204	86.9	10.3	57.1	32.6	75.98	10.92
			205	34.6	76.3	23.7	0	43.64	−9.04
			206	37.0	70.9	29.1	0	45.44	−8.44
			207	36.0	71.0	29.0	0	45.44	−9.44
			208	31.8	71.1	28.9	0	45.44	−13.64
Guernsey Reservoir	Wyoming	USBR	209	40.8	73.0	27.0	0	44.72	−3.92
			210	48.6	68.0	32.0	0	46.52	2.08

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Guernsey Reservoir—Continued	Wyoming	USBR	211	59.6	41.0	55.0	4.0	57.28	2.32
			212	71.6	29.0	60.0	11.0	63.42	8.18
			213	70.0	41.0	55.0	4.0	57.28	12.72
			214	74.4	29.0	64.0	7.0	62.38	12.02
			215	53.9	44.0	53.0	3.0	55.94	−2.04
			216	82.3	39.0	56.0	5.0	58.26	24.04
Morena Reservoir	California	City of San Diego	217	76.9	0	15.0	85.0	93.10	−16.20
			218	93.8	0	3.0	97.0	96.22	−2.42
			219	72.5	0	22.0	78.0	91.28	−18.78
			220	69.4	0	37.0	63.0	87.38	−17.98
			221	84.4	0	13.0	87.0	93.62	−9.22
			222	95.0	2.0	2.0	96.0	95.24	−.24
			223	96.3	3.0	1.0	96.0	94.88	1.42
			224	71.9	1.0	6.0	93.0	94.82	−22.92
			225	83.1	0	14.0	86.0	93.36	−10.26
			226	69.4	0	39.0	61.0	86.86	−17.46
			227	93.1	0	1.0	99.0	96.74	−3.64
			228	91.3	0	25.0	75.0	90.50	.80
			229	81.3	0	7.0	93.0	95.18	−13.88
			230	90.0	0	0.0	100.0	97.00	−7.00
Fern Ridge Reservoir	Oregon	CE	231	54.0	35.0	60.0	5.0	59.70	−5.70
			232	56.0	32.0	62.0	6.0	61.04	−5.04
Arrowrock Reservoir	Idaho	USBR	233	53.9	13.0	54.0	33.0	74.90	−21.00
			234	87.9	1.0	7.0	92.0	94.56	−6.66
			235	60.4	6.0	24.0	70.0	87.04	−26.64
			236	44.2	41.0	49.0	10.0	57.84	−13.64
			237	102.2	1.0	4.0	95.0	94.34	7.66
			238	64.2	13.0	26.0	61.0	82.18	−17.98
			239	52.7	25.0	54.0	21.0	67.46	−14.76
			240	61.5	13.0	68.0	19.0	71.26	−9.76
			241	85.2	1.0	2.0	97.0	95.86	−10.66
			242	47.7	10.0	56.0	34.0	76.24	28.54
			243	57.9	11.0	61.0	28.0	74.32	−16.42
			244	52.3	14.0	60.0	26.0	72.72	−20.42
			245	58.8	10.0	46.0	44.0	78.84	−20.04
			246	57.3	10.0	60.0	30.0	75.20	−17.90
			247	48.3	10.0	56.0	34.0	76.24	−27.94
			248	112.5	1.0	4.0	95.0	95.34	17.16
			249	85.9	0	4.0	96.0	95.96	−10.06
			250	52.1	26.0	53.0	21.0	67.10	−15.00
Lake Corpus Christi	Texas	SCS	251	34.0	77.0	19.0	4.0	44.32	−10.32
			252	55.0	68.0	27.0	5.0	47.82	7.18
			253	62.0	39.0	53.0	8.0	59.04	2.96
			254	30.0	87.0	7.0	6.0	41.24	−11.24
			255	33.8	78.0	17.0	5.0	44.22	−10.42
			256	57.8	62.0	23.0	15.0	52.58	5.22
Lake Waco	Texas	SCS	257	42.2	76.0	21.0	3.0	44.42	−2.22
			258	55.2	54.0	32.0	14.0	55.20	0
			259	49.3	60.0	26.0	14.0	53.04	−3.74
			260	42.2	80.0	16.0	4.0	43.24	−1.04
			261	45.6	70.0	22.0	8.0	47.88	−2.28
			262	70.1	57.0	23.0	20.0	55.68	14.42
			263	73.7	32.0	17.0	51.0	72.74	.96
			264	75.9	50.0	27.0	23.0	58.98	16.92
Cold Springs Reservoir	Oregon	SCS	265	64.0	18.0	71.0	11.0	67.38	−3.38
			266	70.0	53.0	31.0	16.0	56.08	13.92
			267	77.0	15.0	31.0	54.0	79.64	−2.64
			268	66.0	18.0	66.0	16.0	68.68	−2.68
			269	72.0	14.0	42.0	44.0	77.40	−5.40
			270	82.0	14.0	47.0	39.0	76.10	5.90
			271	92.0	14.0	52.0	34.0	74.80	17.20
			272	101.0	11.0	8.0	81.0	88.10	12.90
Tongue River Reservoir	Montana	USBR	273	72.0	18.8	47.7	33.5	72.94	−.94
			274	78.5	24.6	45.6	29.8	69.80	8.70
			275	80.6	20.8	46.0	33.2	72.02	8.58
			276	42.6	49.6	49.0	1.4	53.26	−10.66
			277	43.6	63.0	35.5	1.5	48.71	−5.11
			278	64.9	18.9	65.7	15.4	68.06	−3.16

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Tongue River Reservoir—Con.	Montana	USBR	279	74.5	31.5	40.5	28.0	66.94	7.56
			280	83.0	21.6	49.3	29.1	70.62	12.38
			281	37.2	52.1	45.5	2.4	52.80	−15.60
			282	41.5	58.4	40.8	0.8	50.38	−8.88
			283	55.3	39.1	47.5	13.4	60.34	−5.04
			284	62.8	31.8	51.2	17.0	63.90	−1.10
			285	63.6	31.8	52.0	16.2	63.64	−.04
			286	78.7	14.0	45.8	40.2	76.36	2.34
			287	70.0	23.0	57.2	19.8	67.92	2.08
			288	91.8	11.8	32.5	55.7	81.24	10.56
			289	90.4	23.3	47.6	29.1	70.26	20.14
Sheridan County State Lake.	Kansas	USBR	290	50.5	33.7	55.7	10.6	61.62	−11.12
			291	49.5	46.9	51.8	1.3	54.34	−4.84
			292	71.0	13.8	64.5	21.7	71.68	−.68
			293	64.5	24.2	64.4	11.4	65.22	−.72
			294	63.4	26.1	67.3	6.6	63.46	−.06
			295	61.3	27.7	67.8	4.5	62.20	−.90
			296	67.7	21.9	68.7	9.4	65.42	2.28
			297	71.0	21.0	55.8	23.2	69.42	1.58
			298	95.7	0	1.3	99.7	97.00	−1.30
			299	89.2	20.3	64.9	14.8	67.70	21.50
Lake Avalon	New Mexico	USBR	300	82.3	40.8	50.5	8.7	58.58	23.72
			301	81.9	41.9	48.0	10.1	58.48	23.42
			302	76.4	33.2	37.5	29.3	66.66	9.74
			303	84.5	52.1	39.7	8.2	54.36	30.14
			304	68.1	35.7	46.4	17.9	62.72	5.38
			305	86.7	22.7	45.7	31.6	71.04	15.66
			306	77.4	35.6	57.1	7.3	59.86	17.54
John Martin Reservoir.	Colorado	CE	307	41.37	35.0	47.0	18.0	63.08	−21.71
			308	84.89	1.0	59.0	40.0	81.04	3.85
			309	57.77	28.0	58.0	14.0	64.56	−6.79
			310	67.33	27.0	68.0	5.0	62.58	4.75
			311	91.53	1.0	87.0	12.0	73.76	17.77
			312	60.42	28.0	59.0	13.0	64.30	−3.88
			313	37.13	21.0	58.0	21.0	68.90	−31.77
			314	62.54	37.0	55.0	8.0	59.76	2.78
			315	83.97	3.0	56.0	41.0	80.58	3.39
			316	80.89	14.0	64.0	22.0	71.68	9.21
			317	53.15	32.0	63.0	5.0	60.78	−7.63
			318	91.50	11.0	41.0	48.0	79.52	11.98
			319	61.62	39.0	59.0	2.0	57.48	4.14
			320	81.50	19.0	74.0	7.0	65.98	15.52
			321	80.57	11.0	78.0	11.0	69.90	10.67
			322	87.52	3.0	41.0	56.0	84.48	3.04
			323	97.07	0	8.0	92.0	94.92	2.15
			324	79.3	6.0	42.0	52.0	82.36	−3.06
			325	73.6	14.0	68.0	18.0	70.64	2.96
			326	65.8	16.0	63.0	21.0	70.70	−4.90
			327	96.1	4.0	34.0	62.0	85.68	10.42
			328	97.6	5.0	40.0	55.0	83.50	14.10
			329	89.3	10.0	51.0	39.0	77.54	11.76
			330	114.0	5.0	32.0	63.0	85.58	28.42
			331	89.7	6.0	36.0	58.0	83.92	5.78
			332	80.5	11.0	49.0	40.0	77.44	3.06
			333	102.5	1.0	8.0	91.0	94.30	8.20
			334	70.6	15.0	47.0	38.0	75.48	−4.88
			335	69.2	21.0	72.0	7.0	65.26	3.94
			336	97.3	5.0	40.0	55.0	83.50	13.80
			337	105.2	1.0	10.0	89.0	93.78	11.42
			338	101.6	2.0	8.0	90.0	93.68	7.92
			339	79.6	15.0	84.0	1.0	65.86	13.74
			340	102.9	1.0	2.0	97.0	95.86	7.04
			341	47.8	65.0	34.0	1.0	47.86	−.06
			342	41.9	60.0	37.0	3.0	50.18	−8.28
			343	40.1	58.0	39.0	3.0	50.90	−10.80
			344	44.6	60.0	37.0	3.0	50.18	−5.58
			345	38.5	63.0	35.0	2.0	48.84	−10.34
			346	43.9	63.0	36.0	1.0	48.58	−4.68

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
John Martin Reservoir—Con.	Colorado	CE	347	44.9	39.0	55.0	6.0	58.52	−13.62
			348	41.6	58.0	40.0	2.0	50.64	−9.04
			349	43.1	49.0	51.0	0	53.36	−10.26
			350	42.6	45.0	53.0	2.0	55.32	−12.72
			351	41.2	71.0	29.0	0	45.44	−4.24
McMillan Reservoir	New Mexico	USBR	352	57.8	71.8	25.1	3.1	45.86	11.94
			353	64.8	73.5	24.7	1.8	45.01	19.79
			354	60.3	77.5	21.1	1.4	43.46	16.84
			355	63.9	73.2	25.8	1.0	44.98	18.92
			356	63.0	75.4	23.6	1.0	44.26	18.74
			357	67.9	79.0	20.9	.1	42.56	25.34
			358	76.9	68.1	29.4	2.5	47.13	29.77
			359	55.8	77.6	22.4	0	42.92	12.88
			360	60.5	72.1	25.8	2.1	45.60	14.90
			361	60.6	70.3	29.7	0	45.80	14.80
			362	62.4	73.9	26.1	0	44.36	18.04
			363	63.1	73.1	25.9	1.0	44.98	18.12
			364	51.2	62.8	37.1	.1	48.32	2.88
			365	57.4	85.5	14.3	.2	40.27	17.13
			366	60.5	72.8	25.7	1.5	45.18	15.32
			367	72.4	79.9	18.0	2.1	42.72	29.68
			368	56.2	78.9	20.6	.5	42.73	13.47
			369	61.8	70.7	28.8	.5	45.68	16.12
			370	63.6	67.1	32.7	.2	46.88	16.72
			371	65.5	54.2	44.4	1.4	51.82	13.68
			372	64.3	63.2	36.2	.6	48.58	15.72
Settling Basin Socorro Main Canal.	New Mexico	USBR	373	88.6	17.7	74.9	7.4	66.34	22.26
			374	75.5	10.4	38.2	51.4	80.66	−5.16
			375	76.8	9.5	16.6	73.9	86.79	9.99
			376	73.6	25.5	66.2	8.3	63.98	9.62
			377	84.9	12.1	28.9	59.0	82.02	2.88
			378	79.9	17.2	43.2	39.6	75.28	4.62
			379	59.3	4.0	14.2	81.8	90.88	−31.58
			380	81.1	22.7	58.1	19.2	67.66	13.44
			381	68.0	13.4	36.3	50.3	79.32	−11.32
			382	71.8	9.3	21.3	69.4	85.70	−13.90
			383	86.7	18.6	64.3	17.1	68.58	18.12
			384	78.6	14.1	43.5	42.4	76.88	1.72
			385	71.1	7.5	9.5	83.0	89.88	−18.78
			386	87.4	3.9	27.5	68.6	87.50	−.10
			387	81.7	6.8	2.7	90.5	92.08	−10.38
			388	81.1	5.8	13.6	80.6	89.90	−8.80
			389	63.0	9.1	14.4	76.5	87.61	−24.61
			390	96.1	5.3	6.5	88.2	92.08	4.02
			391	68.6	53.0	46.4	0.6	52.18	16.42
			392	76.1	20.2	20.3	59.5	79.20	−3.10
			393	55.5	26.6	69.4	4.0	62.32	−6.82
			394	74.3	12.0	12.4	75.6	86.44	−12.44
			395	67.4	16.4	29.8	53.8	79.28	−11.88
			396	68.6	26.2	58.5	15.3	65.54	3.06
			397	88.6	9.6	56.1	34.3	76.24	12.36
			398	76.8	18.7	57.4	23.9	70.40	6.40
			399	64.9	34.4	47.2	18.4	63.44	1.46
			400	84.2	26.4	47.6	26.0	68.40	15.80
			401	68.0	32.7	47.3	20.0	64.32	3.68
			402	60.5	10.1	13.7	76.2	87.16	−26.66
			403	94.2	5.3	8.3	86.4	91.56	2.64
			404	74.9	7.5	10.9	81.6	89.52	−14.62
			405	69.3	8.9	18.2	72.9	86.74	−17.44
			406	86.1	21.2	55.0	23.8	69.68	16.42
			407	74.9	13.0	28.1	58.9	81.66	−6.76
			408	69.3	8.7	10.6	80.7	88.82	−19.52
			409	79.9	25.0	53.8	21.2	67.46	12.44
			410	74.9	18.0	35.0	47.0	76.74	−1.84
			411	57.4	9.5	15.7	74.8	87.03	−29.63
			412	61.2	9.2	13.0	77.8	88.04	−26.84
			413	85.5	24.2	56.8	19.0	67.30	18.20
			414	65.5	13.4	25.4	61.2	82.18	−16.68

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Lake Sabetha	Kansas	ARS	415	33.4	73.8	26.1	.1	44.36	−10.96
			416	47.8	54.8	44.9	.3	51.20	−3.40
			417	41.8	65.0	34.9	.1	47.60	−5.80
			418	59.9	37.9	61.8	.3	57.32	2.58
			419	63.8	27.6	70.4	2.0	61.44	2.36
			420	50.4	47.5	51.7	.8	54.11	−3.71
			421	65.1	31.3	62.3	6.4	61.40	3.70
			422	74.9	39.5	57.0	3.5	57.69	17.21
			423	76.4	27.6	71.3	1.1	61.18	15.22
			424	73.5	24.4	74.7	.9	62.62	10.88
			425	71.4	23.3	75.8	.9	62.98	8.42
			426	82.7	22.3	76.5	1.2	63.34	19.36
			427	46.7	86.9	9.5	3.6	40.72	5.98
			428	46.1	86.7	10.3	3.0	40.46	5.64
			429	38.1	88.9	8.1	3.0	39.74	−1.64
			430	40.5	86.7	9.5	3.8	40.72	−.22
			431	40.5	87.5	8.2	4.3	40.62	−.12
			432	39.3	85.7	10.4	3.9	41.08	−1.78
			433	41.2	82.5	7.3	10.2	43.95	−2.75
			434	47.4	80.3	12.4	7.3	44.02	3.38
			435	37.4	89.0	7.5	3.5	39.87	−2.47
			436	42.4	83.7	10.2	6.1	42.32	.08
			437	49.3	82.1	10.5	7.4	43.30	6.00
			438	51.1	79.2	13.9	6.9	44.38	6.72
			439	47.4	93.8	3.0	3.2	37.94	9.46
			440	65.5	70.7	18.3	11.0	48.30	17.20
			441	68.6	68.1	18.1	13.8	50.16	18.44
			442	68.6	68.7	19.5	11.8	49.28	19.32
			443	56.8	75.4	19.6	5.0	45.30	11.50
			444	63.0	76.2	16.9	6.9	45.46	17.54
			445	39.9	72.6	18.2	9.2	47.06	−7.16
			446	71.8	71.7	17.6	10.7	47.94	23.86
			447	56.1	69.4	18.8	11.8	49.28	6.82
			448	81.1	66.9	19.6	13.5	50.43	30.67
			449	80.5	62.6	17.5	19.9	53.52	26.98
			450	39.9	74.6	14.9	10.5	46.87	−6.97
			451	49.9	75.2	15.6	9.2	46.34	3.56
			452	44.3	80.6	14.8	4.6	43.14	1.16
			453	48.0	75.1	14.3	10.6	46.86	1.14
			454	53.0	78.8	14.3	6.9	44.38	8.62
			455	48.7	80.3	17.2	2.5	42.74	5.96
			456	68.0	71.2	19.0	9.8	48.04	19.96
			457	78.0	58.0	11.4	30.6	58.18	19.82
			458	75.0	66.8	18.9	14.3	50.52	24.48
			459	40.5	84.3	11.0	4.7	42.06	−1.56
			460	31.2	84.6	9.5	5.9	41.96	−10.76
			461	76.8	67.1	18.1	14.8	50.78	26.02
			462	86.9	60.3	16.8	22.9	55.38	31.52
TYPE III.—RESERVOIR OPERATION: RESERVOIR NORMALLY EMPTY									
Lake Claremore	Oklahoma	SCS	1	43.0	76.0	22.9	1.1	47.93	−4.93
			2	45.0	72.4	27.4	.2	48.96	−3.96
			3	40.0	71.8	27.7	.5	49.15	9.15
			4	44.0	61.0	38.8	.2	52.48	−8.48
			5	51.0	41.7	57.9	.4	58.56	−7.56
			6	55.0	45.4	53.4	1.2	57.85	−2.85
			7	54.0	39.9	59.0	1.1	59.45	−5.45
			8	63.0	32.1	65.1	2.8	62.51	.49
			9	51.0	50.2	49.4	.4	56.00	5.00
			10	65.0	24.6	53.8	21.6	69.50	−4.50
Grisham Lake	Missouri	SCS	11	53.0	28.6	67.4	4.0	63.72	−10.72
			12	56.0	25.1	66.8	8.1	66.00	−10.00
			13	117.0	0	2.2	97.8	96.50	20.50
High Point Reservoir	North Carolina	SCS	14	59.0	24.3	53.6	22.1	69.82	−10.82
			15	62.0	20.4	56.6	23.0	71.35	−9.35
			16	68.0	68.6	31.0	.4	49.92	18.08
			17	41.0	54.0	45.0	1.0	54.97	−13.97

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
High Point Reservoir Continued	North Carolina	SCS	18	60.0	31.8	65.6	2.6	62.51	−2.51
			19	44.0	28.6	66.0	5.4	63.97	−19.97
			20	64.0	76.7	23.0	.3	47.36	16.64
			21	45.0	25.6	62.1	12.3	66.68	−21.68
			22	37.0	41.8	56.8	1.4	58.81	−21.81
			23	40.0	83.1	15.3	1.6	45.94	−5.94
Kirk Lake	Kansas	SCS	24	42.0	63.1	35.8	1.1	52.09	−10.09
			25	55.0	44.6	53.8	1.6	58.10	−3.10
Lake Lee	North Carolina	SCS	26	62.0	55.9	41.4	2.7	54.83	7.17
			27	60.0	59.3	39.0	1.7	53.62	6.38
			28	59.0	49.5	49.7	.8	56.36	2.64
			29	59.0	37.3	60.8	1.9	60.66	−1.66
			30	60.0	42.8	55.3	1.9	58.74	1.26
			31	61.0	40.6	58.0	1.4	59.13	1.87
			32	66.0	40.8	58.1	1.1	59.13	6.87
			33	73.0	28.8	69.2	2.0	63.22	9.78
Lake Marinuka	Wisconsin	SCS	34	40.0	63.8	34.8	1.4	51.77	−11.77
			35	52.0	67.7	32.3	0	50.24	1.76
			36	56.0	40.6	59.1	.3	58.88	−2.88
			37	55.0	39.9	59.8	.3	59.20	−4.20
			38	60.0	30.8	68.6	.6	62.33	−2.33
			39	70.0	33.3	63.5	3.2	62.19	−7.81
			40	63.0	36.9	62.8	.5	60.32	2.68
			41	60.0	29.4	69.5	1.1	62.97	−2.97
			42	66.0	23.1	72.3	4.6	65.89	.11
			43	80.0	11.7	70.6	16.7	72.41	7.59
			44	77.0	18.8	73.5	7.7	67.92	9.08
			45	69.0	18.8	64.1	17.1	70.17	−1.17
			46	85.0	9.2	68.4	22.4	74.62	10.38
			47	87.0	16.5	72.3	11.2	69.52	17.48
Shepherd Mt. Lake	Missouri	SCS	48	43.0	20.0	64.8	15.2	69.35	−26.35
			49	85.0	0	1.4	99.6	97.00	−12.00
Moran Reservoir	Kansas	SCS	50	50.0	62.8	36.8	.4	51.84	−1.84
			51	38.0	69.0	30.3	.7	50.17	−12.17
			52	52.0	46.5	52.4	1.1	57.40	−5.40
			53	62.0	24.6	56.1	19.3	68.75	−6.75
			54	49.0	41.6	57.3	1.1	58.81	−9.81
			55	43.0	65.0	34.7	.3	51.20	−8.20
Mountain Lake	Missouri	SCS	56	57.0	33.0	62.2	4.8	62.69	−5.69
			57	66.0	18.7	78.2	3.1	66.67	−0.67
Neosho County State Lake.	Kansas	SCS	58	37.0	74.8	23.2	2.0	48.50	−11.50
			59	30.0	64.3	34.1	1.6	52.02	−22.02
			60	36.0	43.9	55.4	.7	58.17	−22.17
			61	38.0	47.8	49.9	2.3	57.14	−19.14
			62	45.0	27.7	68.5	3.8	64.04	−19.04
Grand Saline	Texas	SCS	63	40.0	67.0	32.0	1.0	50.81	−10.81
			64	35.0	68.0	30.0	2.0	50.74	−15.74
			65	47.0	33.0	53.0	14.0	64.94	−17.94
			66	30.0	60.0	34.0	6.0	54.30	−24.30
			67	34.0	54.0	43.0	3.0	55.47	−21.47
			68	34.0	58.0	40.0	2.0	53.94	−19.94
			69	47.0	34.0	59.0	7.0	62.87	−15.87
			70	43.0	43.0	50.0	7.0	59.99	−16.99
Wills Point Reservoir	Texas	SCS	71	51.0	52.0	46.0	2.0	55.86	−4.86
			72	53.0	55.0	42.0	3.0	55.15	−2.15
			73	53.0	50.0	48.0	2.0	56.50	−3.50
			74	59.0	54.0	45.0	1.0	54.97	4.03
			75	54.0	60.0	38.0	2.0	53.30	0.70
			76	86.0	26.0	57.0	17.0	67.93	18.07
			77	85.0	26.0	52.0	22.0	69.18	15.82
			78	45.0	73.0	26.0	1.0	48.89	−3.89
			79	52.0	71.0	27.0	2.0	49.78	2.22
Miami Cons. Dist. Germantown Res.		SCS	80	80.0	29.0	71.0	0	62.72	17.28
			81	79.0	30.0	68.0	2.0	62.90	16.10
Lake Arthur	South Africa		82	38.0	73.0	27.0	0	48.64	−10.64
			83	39.0	61.0	39.0	0	52.48	−13.48
			84	58.0	61.0	39.0	0	52.48	5.52
			85	100.0	3.0	13.0	84.0	92.04	7.96

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Lake Arthur—Continued	South Africa		86	72.0	36.0	35.0	29.0	67.73	4.27
			87	64.0	46.0	51.0	3.0	58.03	5.97
			88	100.0	3.0	19.0	78.0	90.54	9.46
			89	80.0	20.0	51.0	29.0	72.85	7.15
			90	100.0	2.0	72.0	26.0	77.86	22.14
Colorado River above Laguna Dam	Arizona	USBR	91	86.0	9.0	31.0	60.0	84.12	1.88
			92	83.0	2.0	14.0	84.0	92.36	−9.36
			93	88.0	3.0	13.0	84.0	92.04	−4.04
			94	81.0	9.0	60.0	31.0	76.87	4.13
			95	86.0	8.0	50.0	42.0	79.94	6.06
			96	72.0	24.0	60.0	26.0	70.82	1.18
			97	87.0	0	10.0	90.0	94.50	−7.50
			98	64.0	30.0	55.0	15.0	66.15	−2.15
Sirianni Lake	Texas	SCS	99	61.2	23.0	15.0	62.0	80.14	−18.94
			100	30.0	69.0	26.0	5.0	51.17	−21.17
Smith Pond	Texas	SCS	101	28.7	69.0	29.0	2.0	50.42	−21.72
			102	26.8	71.0	27.0	2.0	49.78	−22.98
XGZ Ranch Pond	Texas	SCS	103	55.5	63.0	36.0	1.0	52.09	3.41
			104	71.8	51.0	46.0	3.0	56.43	15.37
Harold Fress Pond	Texas	SCS	105	84.9	41.0	47.0	12.0	61.88	23.02
			106	54.3	37.0	51.0	12.0	63.16	−8.86
Lake Eddleman	Texas	SCS	107	34.5	36.0	14.0	0	60.48	−25.98
			108	36.4	60.0	39.0	1.0	53.05	−16.65
			109	52.7	41.0	55.0	4.0	59.88	−7.18
White Rock Lake	Texas	SCS	110	30.6	91.6	9.0	0	42.88	−12.28
			111	32.4	92.0	8.0	0	42.56	−10.16
			112	31.8	91.0	9.0	0	42.88	−11.08
			113	40.5	91.0	9.0	0	42.88	−2.38
			114	41.3	85.0	15.0	0	44.80	−3.50
Lake Crook	Texas	SCS	115	30.7	83.0	16.0	1	45.69	−14.99
			116	43.0	85.0	15.0	0	44.80	−1.80
			117	27.5	89.0	11.0	0	43.52	−16.02
			118	44.5	67.0	33.0	0	50.56	−6.06
Dawson City Lake	Texas	SCS	119	29.7	82.0	18.0	0	45.76	−16.06
			120	32.2	78.0	22.0	0	47.04	−14.84
			121	36.7	69.0	30.0	1.0	50.17	−13.47
Kerens City Lake	Texas	SCS	122	76.1	19.0	26.0	55.0	79.67	−3.57
			123	59.2	51.0	46.0	3.0	56.43	2.77
			124	33.8	70.0	28.0	2.0	50.10	−16.30
			125	48.9	52.0	38.0	10.0	57.86	−8.96
Loring Lake	Louisiana	SCS	126	81.5	15.0	31.0	54.0	80.70	.80
			127	89.6	12.0	37.0	51.0	80.91	8.69
			128	74.0	8.0	11.0	81.0	89.69	−15.69
			129	92.2	6.0	20.0	74.0	88.58	3.62
			130	88.0	8.0	24.0	68.0	86.44	1.56
Jenkins Pond	Louisiana	SCS	131	69.4	14.0	23.0	63.0	83.27	−13.87
			132	88.0	9.0	19.0	72.0	87.12	.88
			133	79.5	8.0	15.0	77.0	88.69	−9.19
Sandstone Creek Structure No. 5.	Oklahoma	SCS	134	83.0	24.0	37.0	39.0	74.07	8.93
Structure No. 6.			135	68.6	37.0	40.0	23.0	65.91	2.69
			136	92.0	18.0	23.0	59.0	80.99	11.01
			137	86.7	17.0	17.0	66.0	83.06	3.64
			138	99.3	1.0	0.0	99.0	96.43	2.87
			139	77.2	19.0	39.0	42.0	76.42	.78
Sandstone Creek Structure No. 9.		SCS	140	67.4	41.0	48.0	11.0	61.63	5.77
			141	70.3	42.0	52.0	6.0	60.06	10.24
			142	87.9	15.0	29.0	56.0	81.20	6.70
			143	79.4	12.0	17.0	71.0	85.91	−6.51
			144	73.9	17.0	30.0	53.0	79.81	−5.91
Sandstone Creek Structure No. 16.			145	68.5	34.0	47.0	19.0	65.87	2.63
			146	99.5	15.0	39.0	46.0	78.70	20.80
			147	62.9	25.0	42.0	33.0	72.25	−9.25
			148	70.4	26.0	44.0	30.0	71.18	−.78
			149	83.2	17.0	45.0	38.0	76.06	7.14
			150	83.2	15.0	25.0	60.0	82.20	1.00
Sandstone Creek Structure No. 22.			151	74.4	18.0	47.0	35.0	74.99	−.59
			152	80.7	26.0	61.0	13.0	66.93	13.77
			153	78.8	24.0	53.0	23.0	70.07	8.73

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Sandstone Creek Structure No. 1.			154	45.7	42.0	49.0	9.0	60.81	−15.11
			155	54.9	30.0	56.0	14.0	65.90	−11.00
			156	75.2	34.0	60.0	6.0	62.62	12.58
			157	80.8	22.0	58.0	20.0	69.96	10.84
Sandstone Creek Structure No. 14			158	61.7	26.0	54.0	20.0	68.68	−6.98
			159	65.6	24.0	47.0	29.0	71.57	−5.79
			160	76.5	17.0	43.0	40.0	76.56	−.06
			161	78.2	17.0	53.0	30.0	74.06	4.14
Sandstone Creek Structure No. 17			162	90.8	9.0	8.0	83.0	89.87	.93
			163	83.2	9.0	35.0	56.0	83.12	.08
			164	84.6	27.0	43.0	30.0	70.86	13.74
			165	60.8	30.0	51.0	19.0	67.15	−6.35
			166	58.7	46.0	38.0	16.0	61.28	−2.58
Lake Fort, Phantom Hill.	Texas	SCS	167	29.3	82.0	17.0	1.0	46.01	−16.71
			168	31.6	74.0	25.0	1.0	48.57	−16.97
			169	29.9	76.0	23.0	1.0	47.93	−18.03
			170	42.4	42.0	52.0	6.0	60.06	−17.66
			171	28.4	82.0	71.0	1.0	46.01	−17.61
			172	32.6	80.0	20.0	0	46.40	−13.80
			173	44.3	44.0	45.0	11.0	60.67	−16.37
			174	50.5	39.0	50.0	11.0	62.27	−11.77
			175	41.4	41.0	53.0	6.0	60.38	−18.98
			176	29.0	73.0	25.0	2.0	49.14	−20.14
			177	36.4	49.0	48.0	3.0	57.07	−20.67
Lake Throckmorton	Texas	SCS	178	26.7	59.0	39.0	2.0	53.62	−26.92
			179	33.0	60.0	39.0	1.0	53.05	−20.05
			180	32.9	59.0	40.0	1.0	53.37	−20.47
			181	31.3	62.0	37.0	1.0	52.41	−21.11
			182	33.8	56.0	42.0	2.0	54.58	−20.78
George Johnson Pond No. P-2.	South Dakota	SCS	183	79.3	53.0	25.0	22.0	60.54	18.76
			184	72.5	54.0	28.0	18.0	59.22	13.28
			185	86.1	37.0	29.0	34.0	68.66	17.44
			186	82.5	45.0	28.0	27.0	64.35	18.15
			187	84.3	43.0	26.0	31.0	65.99	18.31
Bertha Armour Pond No. P-8.	South Dakota	SCS	188	81.1	45.0	23.0	32.0	65.60	15.50
			189	78.6	31.0	29.0	40.0	72.08	6.62
			190	93.6	27.0	28.0	45.0	74.61	18.99
			191	91.9	16.0	29.0	55.0	80.63	11.27
Charles Crago	South Dakota	SCS	192	41.8	77.0	16.0	7.0	49.11	−7.31
Charles Crago Pond No. P-10.	South Dakota	SCS	192	41.8	77.0	16.0	7.0	49.11	−7.31
			193	29.3	84.0	13.0	3.0	45.87	−16.57
			194	29.9	83.0	13.0	4.0	46.44	−16.54
			195	71.1	66.0	20.0	14.0	54.38	16.72
			196	58.0	81.0	14.0	5.0	47.33	10.67
Charles Crago Pond No. P-11.	South Dakota	SCS	197	39.9	85.0	9.0	6.0	46.30	−6.40
			198	47.4	79.0	18.0	3.0	47.47	−.07
			199	64.9	87.0	8.0	5.0	45.41	19.49
			200	36.2	88.0	11.0	1.0	44.09	−7.89
			201	53.6	90.0	9.0	1.0	43.45	10.15
			202	43.7	85.0	12.0	3.0	45.55	−1.85
			203	48.0	82.0	15.0	3.0	46.51	1.49
M. I. Hefner Pond No. P-12.	South Dakota	SCS	204	67.5	77.0	18.0	5.0	48.61	18.89
			205	80.0	69.0	15.0	16.0	53.92	26.08
Ashland Reservoir	Missouri	SCS	206	84.0	4.0	51.0	45.0	81.97	2.03
			207	80.0	2.0	48.0	50.0	83.86	−3.86
			208	82.0	4.0	78.0	18.0	75.22	6.78
			209	67.0	5.0	82.0	13.0	73.65	−6.65
			210	62.0	5.0	86.0	9.0	72.65	−10.65
			211	61.0	8.0	83.0	9.0	71.69	−10.69
			212	54.0	26.0	64.0	10.0	66.18	−12.18
			213	85.0	5.0	69.0	26.0	76.90	8.10
			214	80.0	8.0	77.0	15.0	73.19	6.81
			215	78.0	8.0	79.0	13.0	72.69	5.31
			216	69.0	11.0	75.0	14.0	71.98	−2.98
			217	62.0	12.0	75.0	13.0	71.41	−9.41
			218	57.0	19.0	63.0	18.0	70.42	−13.42
			219	55.0	29.0	61.0	10.0	65.22	−10.22
			220	57.0	28.0	61.0	11.0	65.79	−8.79

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Pond No. 1	Illinois	Illinois State Water Survey.	221	51.79	75.7	42.2	.1	47.68	4.11
			222	65.52	63.7	21.9	14.4	55.02	10.20
			223	66.77	56.9	21.1	22.0	59.26	7.51
			224	39.31	80.8	18.3	.9	46.33	−7.02
			225	60.53	77.0	21.7	1.3	47.61	12.92
			226	59.28	80.0	17.1	2.9	47.15	12.13
			227	69.26	65.2	27.1	7.7	53.20	16.06
			228	62.40	71.6	23.4	5.0	50.21	12.19
			229	67.39	64.7	24.8	10.5	53.92	13.47
			230	73.63	65.2	21.4	13.4	54.45	19.18
			231	53.66	76.7	21.7	1.6	47.86	5.80
			232	74.26	64.0	20.6	15.4	55.27	18.99
Pond No. 4	Illinois	Illinois State Water Survey.	233	48.67	79.4	19.2	1.4	46.97	1.70
			234	32.45	85.0	14.3	.7	45.05	−12.60
			235	68.64	84.5	14.5	1.0	45.21	23.43
			236	40.56	78.7	18.2	3.1	47.47	−6.91
			237	76.13	52.0	15.5	32.5	63.48	12.65
Pond No. 5	Illinois	Illinois State Water Survey.	238	59.28	65.7	22.4	11.9	53.88	5.40
			239	67.39	74.3	21.2	4.5	49.35	18.04
			240	55.54	78.1	21.2	.7	47.29	8.25
			241	39.31	79.3	20.3	.4	46.72	−7.41
			242	62.40	71.8	27.5	.7	49.21	13.19
			243	42.43	82.0	17.5	.5	45.88	−3.45
Pond No. 6	Illinois	Illinois State Water Survey.	244	38.06	80.4	18.7	.9	46.65	−8.59
			245	53.66	79.8	19.3	.9	46.65	7.01
			246	68.02	75.1	22.8	2.2	48.50	19.52
			247	56.78	70.1	28.2	1.7	50.10	6.68
			248	54.28	74.1	22.8	3.1	49.07	5.21
			249	70.51	65.9	22.3	11.8	53.88	16.63
			250	66.70	70.4	23.9	5.7	51.10	15.60
Pond No. 8	Illinois	Illinois State Water Survey.	251	48.67	10.7	13.6	5.7	47.58	1.09
			252	48.67	79.8	15.2	5.0	47.65	1.02
			253	59.90	75.1	24.3	.6	48.25	11.65
			254	69.89	76.4	21.4	2.2	48.18	21.71
			255	64.27	64.8	24.9	10.3	53.70	10.57
			256	58.65	81.8	17.5	.7	46.01	12.64
			257	50.54	70.5	23.6	5.9	50.92	−.38
Pond No. 9	Illinois	Illinois State Water Survey.	258	66.14	67.3	24.1	8.6	52.81	13.33
			259	66.77	64.5	32.7	2.8	52.06	14.71
			260	42.38	62.9	29.1	8.0	53.84	−11.46
Pond No. 15	Illinois	Illinois State Water Survey.	261	46.80	81.9	17.9	.2	45.76	1.04
			262	61.77	74.5	22.2	3.3	48.98	12.79
			263	52.41	74.2	24.5	1.3	48.57	3.84
			264	54.29	75.0	24.6	.4	48.00	6.29
			265	63.02	76.9	22.6	.5	47.52	15.50
			266	61.77	74.8	22.7	.5	48.19	13.58
Pond No. 16	Illinois	Illinois State Water Survey.	267	29.33	78.0	18.3	3.7	48.04	−18.71
			268	29.33	73.8	25.8	.4	48.32	−18.99
			269	44.93	80.4	17.9	1.7	46.90	−1.97
			270	58.03	71.8	24.4	3.8	49.96	8.07
			271	58.03	76.4	22.3	1.3	47.93	10.10
			272	28.08	80.9	17.8	1.3	46.33	−18.25
			273	45.55	78.9	19.9	1.2	46.97	−1.42
			274	49.30	74.9	22.3	2.8	48.75	.55
			275	30.58	85.6	14.3	.1	44.48	−13.90
			276	48.67	50.2	12.5	37.3	65.25	−16.58
			277	43.06	79.1	19.0	1.9	47.22	−4.16
			278	30.58	78.0	16.4	5.6	48.54	−17.96
Pond No. 17	Illinois	Illinois State Water Survey.	279	48.67	86.2	13.0	.8	44.73	3.94
			280	44.30	84.9	14.6	.5	44.96	−.66
			281	41.81	76.0	23.5	.5	47.80	−5.99
			282	45.55	87.5	12.0	.5	44.12	1.43
			283	57.41	73.7	23.3	3.0	49.07	8.34
			284	52.42	67.7	29.9	2.4	50.74	1.68
			285	68.64	64.4	22.9	12.7	54.77	13.87
			286	79.25	62.4	22.8	14.8	55.91	23.34
			287	52.42	69.4	28.0	2.6	50.67	1.75

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Pond No. 20	Illinois	Illinois State Water Survey.	288	34.32	86.1	12.9	1.0	44.73	−10.41
			289	31.82	82.7	16.2	1.1	45.69	−13.87
			290	49.92	80.1	18.7	1.2	46.65	3.27
			291	46.80	74.1	19.7	6.2	49.82	−3.02
			292	36.82	78.9	20.6	.5	46.88	−10.06
			293	46.18	75.6	22.1	2.3	48.18	−2.00
			294	51.17	78.1	21.3	.6	47.29	3.88
			295	77.38	59.3	33.8	6.9	54.87	22.51
			296	63.65	68.0	29.3	2.7	50.99	12.66
			297	63.65	72.8	25.8	1.4	48.89	14.76
Pond No. 21	Illinois	Illinois State Water Survey.	298	38.69	87.0	12.0	1.0	44.41	−5.72
			299	38.69	83.3	16.2	.5	45.47	−6.78
			300	37.44	84.7	14.6	.7	45.05	−7.61
			301	41.18	84.6	14.9	.5	45.05	−3.87
			302	47.42	83.9	15.8	.3	45.1	2.30
			303	62.40	76.2	23.4	.4	47.68	14.72
			304	66.77	74.2	24.6	1.2	48.57	18.20
			305	72.38	67.1	26.7	6.2	52.00	20.32
			306	67.39	73.6	24.3	2.1	48.82	18.57
			307	74.26	65.9	27.6	6.5	52.54	21.72
Pond No. 24	Illinois	Illinois State Water Survey.	308	40.56	66.2	17.2	16.6	55.13	−14.57
			309	31.82	76.3	21.4	2.3	48.18	−16.36
			310	33.69	78.3	18.9	2.8	47.79	−14.10
			311	36.19	73.3	22.3	4.4	49.64	−13.45
			312	34.32	77.0	20.3	2.7	48.11	−13.79
			313	35.57	79.4	18.6	2.0	47.22	−11.65
			314	38.06	72.8	23.0	4.2	49.64	−11.58
			315	46.18	74.7	19.7	5.6	49.50	−3.32
			316	43.68	78.6	19.6	1.8	47.22	−3.54
Pond No. 25	Illinois	Illinois State Water Survey.	317	41.18	72.3	23.1	4.6	50.21	−9.03
			318	66.14	63.8	24.0	12.2	54.52	11.62
			319	70.51	60.6	21.6	17.8	56.98	13.53
			320	39.31	79.4	19.1	1.5	46.97	−7.66
			321	52.42	75.6	22.1	2.3	48.18	4.24
Pond No. 29	Illinois	Illinois State Water Survey.	322	36.19	76.8	22.3	.9	47.61	−11.42
			323	39.94	76.0	21.5	2.5	48.30	−8.36
			324	55.66	73.7	23.0	3.3	49.07	6.59
			325	39.31	79.0	19.5	1.5	47.10	−7.79
			326	48.67	78.5	20.3	1.2	47.18	1.49
			327	76.13	65.6	17.8	16.6	55.13	21.00
			328	73.63	72.7	23.9	3.4	49.39	24.24
Pond No. 33	Illinois	Illinois State Water Survey.	329	61.15	81.2	18.5	.4	46.08	15.07
			330	65.52	80.1	19.2	.7	46.65	18.87
Pond No. 42	Illinois	Illinois State Water Survey.	331	19.97	77.2	21.7	1.1	47.61	−27.64
			332	38.69	73.8	24.1	2.1	48.82	−10.13
			333	29.95	70.2	26.3	3.5	50.41	−20.46
			334	33.70	72.5	21.3	6.2	50.35	−16.65
			335	48.05	70.8	23.0	6.2	50.78	−2.73
			336	50.54	70.4	26.5	3.1	50.35	.19
			337	49.30	72.9	25.3	1.8	49.14	.16
			338	37.44	69.2	29.7	1.1	50.17	−12.73
			339	53.66	72.1	25.0	2.9	49.71	3.95
			340	44.30	64.8	31.1	4.1	52.20	−7.90
Pond No. 44	Illinois	Illinois State Water Survey.	341	41.81	75.6	23.6	.8	47.93	−6.12
			342	51.79	79.9	19.2	.9	46.65	5.14
			343	60.53	69.6	27.9	2.5	50.35	10.18
			344	63.65	68.8	27.3	3.9	50.92	12.73
			345	65.52	76.2	22.0	1.8	48.18	17.34
			346	58.66	59.9	35.3	4.8	54.05	4.61
			347	78.00	66.5	29.4	4.1	51.74	26.26
Pond No. 46	Illinois	Illinois State Water Survey.	348	33.70	86.0	13.0	1.0	44.73	−11.03
			349	40.56	84.7	13.6	1.7	45.30	−4.74
			350	39.94	82.4	16.4	1.2	46.01	−6.07
			351	44.93	84.2	14.3	1.5	45.43	−.50
			352	53.04	79.5	19.7	.8	46.76	6.28
			353	58.66	77.8	20.7	1.5	47.48	11.18

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Thompson Lake	Illinois	Illinois State Water Survey.	354	60.53	76.3	17.1	6.6	49.43	11.10
			355	25.58	70.0	25.2	4.8	50.85	25.27
			356	39.31	71.7	22.9	5.4	50.21	10.90
Lake Pana	Illinois	Illinois State Water Survey.	357	44.93	88.4	11.0	.6	44.09	.84
			358	45.55	90.1	9.3	.6	43.45	2.10
			359	49.92	85.6	13.8	.6	44.73	5.19
			360	38.69	81.0	15.8	3.2	46.83	−8.14
			361	41.81	78.3	20.2	1.5	47.32	−5.51
			362	74.26	76.9	22.7	.4	47.36	26.90
Kahola Reservoir	Kansas	SCS	363	37.9	37.0	51.0	12.0	63.16	−25.26
			364	39.8	36.0	56.0	8.0	62.48	−22.68
			365	39.7	34.0	54.0	12.0	64.12	−24.42
			366	42.9	34.0	58.0	8.0	63.12	−20.22
			367	69.7	21.0	73.0	6.0	66.78	2.92
			368	77.5	14.0	80.0	6.0	69.02	8.48
			369	77.0	18.0	79.0	3.0	66.99	10.01
			370	61.3	25.0	70.0	5.0	65.25	−3.95
			371	52.8	23.0	71.0	6.0	66.14	−13.34
			372	68.1	21.0	68.0	11.0	68.03	.07
			373	70.5	18.0	76.0	6.0	67.74	2.76
			374	36.1	40.0	54.0	6.0	60.70	−24.60
			375	54.8	21.0	66.0	13.0	68.53	−13.73
			376	42.8	36.0	47.0	7.0	62.23	−19.43
			377	50.2	16.0	75.0	9.0	69.13	−18.93
Whitehead Reservoir	Nebraska	SCS	378	60.5	41.0	48.0	11.0	61.63	−1.13
			379	64.4	41.0	50.0	9.0	61.13	3.27
			380	88.6	24.0	33.0	43.0	75.07	13.53
			381	65.5	35.0	44.0	21.0	66.05	−0.55
			382	82.4	34.0	44.0	22.0	66.62	−15.78
			383	71.2	34.0	46.0	20.0	66.12	5.08
			384	78.0	30.0	40.0	30.0	69.90	8.10
			385	55.5	34.0	53.0	13.0	64.37	−8.87
			386	76.8	36.0	44.0	20.0	65.48	−11.32
			387	51.8	39.0	46.0	15.0	63.27	−11.47
			388	49.9	49.0	43.0	8.0	58.32	−8.42
			389	65.0	28.0	57.0	13.0	66.29	−1.29
			390	59.9	31.0	55.0	14.0	65.58	−5.68
			391	61.8	34.0	46.0	20.0	66.12	−4.32
			392	56.8	40.0	49.0	11.0	61.95	−5.15
			393	47.4	32.0	53.0	15.0	65.51	−18.11
			394	69.4	36.0	45.0	19.0	65.23	4.17
			395	74.3	31.0	48.0	21.0	67.33	6.97
Bruce Dempsey Pond	Nebraska	SCS	396	73.0	42.0	40.0	18.0	63.06	9.94
			397	71.8	27.0	45.0	29.0	70.61	1.19
			398	48.0	38.0	46.0	16.0	63.84	−15.84
			399	57.4	30.0	53.0	17.0	66.65	−9.25
			400	83.0	32.0	48.0	20.0	66.76	16.24
			401	75.0	37.0	47.0	16.0	64.16	10.84
			402	80.0	29.0	58.0	13.0	65.97	14.03
			403	75.0	31.0	56.0	13.0	65.33	9.67
			404	80.0	34.0	53.0	13.0	64.37	15.63
			405	79.4	31.0	56.0	13.0	65.33	14.07
TYPE IV.—RIVERBED SEDIMENTS									
Missouri River near Kansas City	Missouri		1	81.0	1.0	29.0	70.0	89.67	−8.67
			2	93.0	0	32.0	68.0	89.32	3.68
			3	97.0	0	5.0	95.0	95.80	1.20
			4	93.0	0	34.0	66.0	88.84	4.16
			5	94.0	1.0	13.0	86.0	93.51	.49
			6	88.0	0	38.0	62.0	87.88	.12
			7	72.0	0	45.0	55.0	86.20	−14.20
			8	86.0	1.0	29.0	70.0	89.67	−3.67
			9	84.0	6.0	77.0	17.0	76.30	7.70
			10	62.0	3.0	86.0	11.0	75.25	−13.25
			11	75.0	1.0	98.0	1.0	73.11	1.89
			12	74.0	4.0	95.0	1.0	72.72	1.28
			13	85.0	2.0	76.0	22.0	78.02	6.98

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Missouri River near Kansas City—Con.	Missouri		14	72.0	3.0	95.0	2.0	73.09	−1.09
			15	87.0	1.0	93.0	6.0	74.31	12.69
			16	72.0	1.0	89.0	10.0	74.27	−2.27
			17	74.0	6.0	88.0	6.0	73.66	.34
			18	98.0	0	25.0	75.0	91.00	7.00
			19	97.0	0	23.0	77.0	91.48	5.52
			20	85.0	0	23.0	77.0	91.48	−6.48
			21	86.0	0	58.0	42.0	83.08	2.92
			22	82.0	1.0	87.0	12.0	75.75	6.25
			23	80.0	0	86.0	14.0	76.36	3.64
			24	88.0	0	79.0	21.0	78.04	9.96
			25	83.0	0	74.0	26.0	79.24	3.76
			26	83.0	0	57.0	43.0	83.32	−.32
			27	84.0	0	65.0	35.0	81.40	2.60
			28	86.0	0	62.0	38.0	82.12	3.88
			29	81.0	0	71.0	29.0	79.96	1.04
			30	85.0	0	50.0	50.0	85.00	0
			31	96.0	0	18.0	82.0	92.68	3.32
			32	88.0	0	10.0	90.0	94.60	−6.60
			33	89.0	0	9.0	91.0	94.84	−5.84
			34	79.0	0	7.0	93.0	95.32	−16.32
			35	86.0	0	8.0	92.0	95.08	−9.08
			36	90.0	0	8.0	92.0	95.08	−5.08
			37	82.0	0	18.0	82.0	92.68	−10.68
			38	86.0	0	18.0	82.0	92.68	−6.68
			39	82.0	0	14.0	86.0	93.64	−11.64
			40	86.0	0	14.0	86.0	93.64	−7.64
			41	52.0	3.0	50.0	47.0	83.89	−31.89
			42	48.0	2.0	45.0	53.0	85.46	−37.46
			43	43.0	10.	39.0	60.0	87.27	−44.27
			44	45.0	2.0	45.0	53.0	85.46	−40.46
			45	66.0	2.0	36.0	62.0	87.62	−21.62
			46	47.0	3.0	37.0	60.0	87.01	−40.01
			47	57.0	1.0	30.0	69.0	89.43	−32.43
			48	63.0	2.0	35.0	63.0	87.86	−24.86
			49	60.0	2.0	34.0	64.0	88.10	−28.10
			50	66.0	1.0	49.0	50.0	84.87	−18.87
Little Colorado River near Grand Falls.	Arizona		51	91.0	0	6.0	94.0	95.56	−4.56
			52	88.0	3.0	55.0	42.0	82.69	5.31
			53	99.0	1.0	24.0	75.0	90.87	8.13
			54	95.0	3.0	17.0	80.0	91.81	3.19
			55	106.0	1.0	9.0	90.0	94.47	11.53
			56	90.0	2.0	12.0	86.0	93.38	−3.38
			57	94.0	2.0	10.0	88.0	93.86	.14
			58	104.0	2.0	9.0	89.0	94.10	9.90
Middle Rio Grande			59	101.0	.3	1.9	97.8	96.43	4.57
			60	86.1	6.0	92.8	11.2	74.86	11.24
			61	86.9	5.4	61.5	33.1	80.27	6.63
			62	73.2	13.9	84.4	1.7	71.66	1.54
			63	103.5	2.6	26.2	71.2	89.65	13.85
			64	81.7	10.3	85.1	4.1	72.53	9.17
			65	76.4	12.0	86.8	1.2	71.68	4.72
			66	90.3	7.1	33.3	59.6	86.49	3.81
			67	79.6	12.8	84.9	2.3	71.79	7.81
			68	102.0	9.1	8.1	82.8	91.75	10.25
			69	87.0	2.9	32.1	65.0	88.21	−1.21
			70	73.0	38.1	61.4	.5	68.17	4.83
			71	83.1	2.4	7.6	90.0	94.34	−11.24
			72	91.4	8.2	72.0	19.8	76.76	14.64
			73	80.2	7.1	62.6	30.3	79.29	.91
			74	98.0	1.5	1.2	97.3	96.16	1.84
			75	87.0	26.6	46.0	27.4	75.97	11.03
			76	48.3	80.3	19.5	.2	62.60	−14.30
			77	94.2	82.9	16.9	.2	62.21	31.99
			78	81.4	14.0	80.7	5.3	72.38	9.02
			79	101.2	4.6	39.6	55.8	85.79	15.41
			80	88.5	15.3	83.7	1.0	71.29	17.21
			81	80.8	34.6	65.1	.3	68.45	12.35

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu. ft.				*Lb./cu. ft.*	
Middle Rio Grande—Continued			82	78.6	36.1	63.7	.2	68.32	10.28
			83	78.5	12.6	85.3	2.1	72.03	6.47
			84	80.9	5.4	58.8	35.8	80.99	−.09
Iowa River near Liberty.	Iowa		85	69.0	24.0	72.0	4.0	70.84	−1.84
			86	61.0	16.0	76.0	8.0	72.84	−11.84
Iowa River near Iowa City.	Iowa		87	56.0	26.0	71.0	3.0	70.34	−14.34
			88	55.0	30.0	67.0	3.0	69.82	−14.82
			89	56.0	18.0	77.0	5.0	71.86	−15.86
Iowa River at Iowa City.	Iowa		90	65.0	12.0	61.0	27.0	77.92	−12.92
			91	64.0	14.0	57.0	29.0	78.14	−14.14
			92	52.0	29.0	67.0	4.0	70.19	−18.19
			93	52.0	10.0	89.0	1.0	71.94	−19.94
			94	67.0	18.0	77.0	5.0	71.86	−4.86
			95	64.0	16.0	78.0	6.0	72.36	−8.36
Cedar River near Cedar Valley.	Iowa		96	90.0	6.0	28.0	66.0	88.06	1.94
			97	109.0	0	0	100.0	97.00	12.00
			98	99.0	0	0	100.0	97.00	2.00
Colorado River at Yuma.	Arizona		99	86.0	2.0	16.0	82.0	92.42	−6.42
			100	88.0	4.0	66.0	30.0	79.68	8.32
			101	84.0	7.0	63.0	30.0	79.29	4.71
			102	93.0	3.0	35.0	62.0	87.49	5.51
			103	69.0	17.0	71.0	12.0	73.67	−4.67
			104	52.0	29.0	50.0	21.0	74.27	−22.27
Colorado River at Yuma, Main Canal.	Arizona		105	87.0	1.0	17.0	82.0	92.55	−5.55
			106	85.0	5.0	55.0	40.0	81.95	3.05
			107	85.0	5.0	57.0	38.0	81.47	3.53
Colorado River below Laguna Dam.	Arizona		108	88.0	4.0	14.0	82.0	92.16	−4.16
			109	87.0	5.0	13.0	82.0	92.03	−5.03
			110	92.0	2.0	14.0	84.0	92.90	−.90
			111	73.0	6.0	25.0	69.0	88.78	−15.78
			112	97.0	2.0	15.0	83.0	92.66	4.34
			113	82.0	19.0	50.0	31.0	77.97	4.03
			114	76.0	20.0	75.0	5.0	71.60	4.40
			115	82.0	12.0	45.0	43.0	81.76	.24
			116	82.0	12.0	38.0	50.0	83.44	−1.44
			117	93.0	1.0	11.0	88.0	93.99	−.99
			118	93.0	1.0	11.0	88.0	93.99	.99
Colorado River at Yuma.	Arizona		119	105.0	0	3.0	97.0	96.28	8.72
			120	98.0	0	3.0	97.0	96.28	1.72
			121	93.0	0	12.0	88.0	94.12	−1.12
			122	89.0	2.0	40.0	48.0	84.26	4.74
			123	95.0	3.0	20.0	77.0	91.09	3.91
			124	78.0	9.0	47.0	44.0	82.39	−4.39
			125	87.0	5.0	30.0	65.0	87.95	−.95
			126	91.0	3.0	20.0	77.0	91.09	−.09
Colorado River above Needles.	California		127	92.0	1.0	48.0	51.0	85.11	6.89
			128	96.0	0	12.0	88.0	94.12	1.88
			129	94.0	0	9.0	91.0	94.84	−.84
			130	95.0	0	6.0	94.0	95.56	−.56
			131	94.0	0	5.0	95.0	95.80	−1.80
			132	93.0	0	3.0	97.0	96.28	−3.28
			133	94.0	0	3.0	97.0	96.28	−2.28
			134	67.0	35.0	62.0	3.0	69.17	−2.17
			135	95.0	1.0	49.0	50.0	84.87	10.13
			136	98.0	0	40.0	60.0	87.40	10.60
			137	90.0	3.0	19.0	78.0	91.33	−1.33
			138	86.0	1.0	60.0	39.0	82.23	3.77
Colorado River near Topock.	Arizona		139	85.0	0	10.0	90.0	94.60	−9.60
			140	83.0	2.0	42.0	56.0	86.18	−3.18
			141	98.0	0	21.0	79.0	91.96	6.04
			142	88.0	1.0	63.0	36.0	81.51	6.49
			143	94.0	1.0	24.0	75.0	90.87	3.13
			144	98.0	1.0	35.0	65.0	88.47	9.53
			145	73.0	2.0	87.0	11.0	75.38	−2.38
			146	82.0	1.0	65.0	34.0	81.03	.97
			147	70.0	4.0	93.0	3.0	73.20	−3.20
			148	98.0	1.0	37.0	72.0	90.15	7.85
			149	88.0	1.0	50.0	49.0	84.63	3.37

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
				Lb./cu.ft.					
Colorado River at Yuma—Continued	Arizona		150	84.0	3.0	57.0	40.0	82.21	1.79
			151	87.0	7.0	65.0	28.0	78.81	8.19
			152	88.0	2.0	54.0	44.0	83.30	4.70
			153	87.0	0	70.0	30.0	80.20	6.80
			154	83.0	12.0	57.0	31.0	78.88	4.12
			155	80.0	2.0	38.0	60.0	87.14	−7.14
			156	87.0	1.0	39.0	60.0	87.27	−.27
			157	83.0	1.0	36.0	63.0	87.99	−4.99
			158	87.0	0	34.0	66.0	88.84	−1.84
			159	87.0	0	33.0	67.0	89.08	−2.08
			160	88.0	1.0	34.0	65.0	88.47	−.47
			161	90.0	0	7.0	93.0	95.32	−5.32
			162	92.0	0	5.0	95.0	95.80	−3.80
			163	83.0	1.0	55.0	44.0	83.43	−.43
			164	80.0	0	44.0	56.0	86.44	−6.44
			165	90.0	0	25.0	75.0	91.00	−1.00
			166	82.0	0	28.0	72.0	90.28	−8.28
			167	80.0	0	18.0	82.0	92.68	−12.68
			168	86.0	0	15.0	85.0	93.40	−7.40
			169	90.0	0	12.0	88.0	94.12	−4.12
			170	98.0	0	10.0	90.0	94.60	3.40
			171	92.0	0	9.0	91.0	94.84	−2.84
			172	78.0	12.0	48.0	40.0	81.04	−3.04
			173	91.0	2.0	20.0	78.0	91.46	−.46
Colorado River near Parker.	Arizona		174	94.0	3.0	70.0	27.0	79.09	14.91
			175	95.0	1.0	37.0	62.0	87.75	7.25
			176	93.0	1.0	59.0	40.0	82.47	10.53
Colorado River near Grand Canyon.	Arizona		177	46.0	30.0	66.0	4.0	70.06	−24.06
			178	87.0	1.0	40.0	59.0	87.03	−.03
			179	88.0	1.0	50.0	49.0	84.63	3.37
			180	87.0	1.0	39.0	60.0	87.27	−.27
			181	84.0	1.0	45.0	54.0	85.83	−1.83
			182	96.0	0	8.0	92.0	95.08	.92
			183	97.0	0	23.0	77.0	91.48	5.52
Colorado River at Lees Ferry.	Arizona		184	100.0	0	14.0	86.0	93.64	6.36
			185	95.0	0	7.0	93.0	95.32	−.32
			186	92.0	1.0	29.0	70.0	89.67	2.33
			187	98.0	0	20.0	80.0	92.20	5.80

operation. For a more accurate guide to the selection, a duration curve of the reservoir levels can be prepared from the data of the reservoir operation study. This duration curve portrays the functional pattern for study and analysis of the reservoir levels.

Theory and Analysis

The determination of average initial unit weight of deposited sediments in Bureau of Reclamation studies generally requires that sediment concentrations be determined on a percentage by weight basis.

Prediction of the unit weight from these percentages demands use of some type of analytical relation. Wherever possible, such a relation should be consistent with physical laws representing the phenomenon described. In many instances, however, these physical laws may be insufficiently understood or of such a complex nature as to preclude their use. Under these circumstances it may be necessary to use some form of empirical relation. Three important characteristics of a relation of this type are:

(1) It should be as simple as consistent with reasonable accuracy.

(2) It should conform as closely as possible to known data.

(3) It should predict reasonable values over the entire range of the variables.

In the discussion below, the notation is as follows:

γ = unit weight;

p = fraction by weight of a given component of a sample;

W = constant determined by regression analysis with dimensions of unit weight, applicable to each component;

c, m, s = subscripts denoting clay, silt, and sand, respectively.

Based on simplicity the linear form of equa-

tion used by Lane and Koelzer [6] is immediately suggested. It is:

$$\gamma = W_c p_c + W_m p_m + W_s p_s \quad (1)$$

A second simple form is the inverse equation:

$$\gamma = \left(\frac{p_c}{W_c} + \frac{p_m}{W_m} + \frac{p_s}{W_s} \right)^{-1} \quad (2)$$

If, in any given sample of material, each size of particle was separately deposited, equation 2 would describe the physical relation between unit weight and fractional weight of the components correctly. However, due to the random nature of the deposition process, particles of all sizes are intermingled and, for this existing situation, equation 2 must be considered as empirical.

In general, intermixing of the particles of various sizes will produce a denser sediment. This suggests use of a form containing products and powers of the various fractions. Quadratic relations similar to equations 1 and 2, including various combinations of the second degree terms in the variables, were therefore considered.

Still another form, which it was believed might help to account for the intermixture of particle sizes, is the exponential form:

$$\gamma = W_c p_c + W_m p_m + W_s p_s + W p_c^i p_m^j p_s^k \quad (3)$$

in which $i + j + k = 1$.

Equations of each of the above described types were applied to sample populations from the data compiled in table 1. The quadratic and exponential forms gave no significant improvement in the fit of the regression equation to the data selected. In addition, their greater complexity increases the labor involved in their computation and use.

Regression equations of types I and II were derived for all of the data of table 1. For each type of reservoir operation the linear equation gave a closer fit to the data, judged by the correlation coefficient and the standard error of the estimate, than the inverse type. This type of equation was, therefore, adopted for representation of the data. Table 2 summarizes the results from the analysis:

TABLE 2.—*Values of coefficients in equation 1 for reservoir types*

Type of reservoir operation	Observations	Values of coefficients in equation 1			R[1]	S[2]
		W_c	W_m	W_s		
	Number					*Lb./cu. ft.*
I	262	26.1	70.2	105.8	0.839	12.0
II	462	27.3	71.4	95.8	.733	14.0
III	405	39.6	67.0	104.3	.729	12.2
IV	187	59.8	73.1	94.3	.572	10.9

[1] R = correlation coefficient.
[2] S = standard error of estimate.

Adopted Procedure

In reviewing the previous tabulation by use of equation 1, it is noted that the sand unit weight coefficient varied from 94.3 to 105.8, or a difference of about 11 pounds per cubic foot. It was then decided that the data of all 1,316 samples be computed in a single run for the purpose of obtaining a representative value of the unit weight coefficient for sand under all types of reservoir operation. These computations resulted in a unit weight coefficient of about 97 pounds per cubic foot for sand. This value was adopted for use in the investigation.

A further inspection of the tabulated values showed a slight decrease in the unit weight coefficient for silt for the type III reservoir operation. This would be contrary to the normal manner by which the sediments physically compact in a reservoir; the value, therefore, was revised upward to 72 pounds per cubic foot. Unit weight coefficients for clay appeared consistent with the normal compaction pattern of sediment accumulation for each type of reservoir operation.

[6] See footnote 4.

Based upon the preceding revision, a different set of values was obtained for each unit weight coefficient. Table 3 summarizes the final values rounded to the nearest pound as determined from this investigation.

The values listed above would consequently revise the correlation coefficients and standard error of estimates as indicated, although neither is greatly changed from those listed in the previous tabulation.

Using the coefficients from the above tabulation, the unit weight of each sample was computed and tabulated in table 1 under the column headed "Computed unit weight." The individual deviations, δ, were also determined from the difference between the observed and computed unit weights. Figures 1 through 4 show graphically the comparison of these observed and computed unit weights for each type of reservoir operation.

The enveloping lines shown in these figures represent the ranges of plus or minus one standard error of estimate and 1.645 times the standard error of estimate indicated, respectively, by the long dashed and short dashed

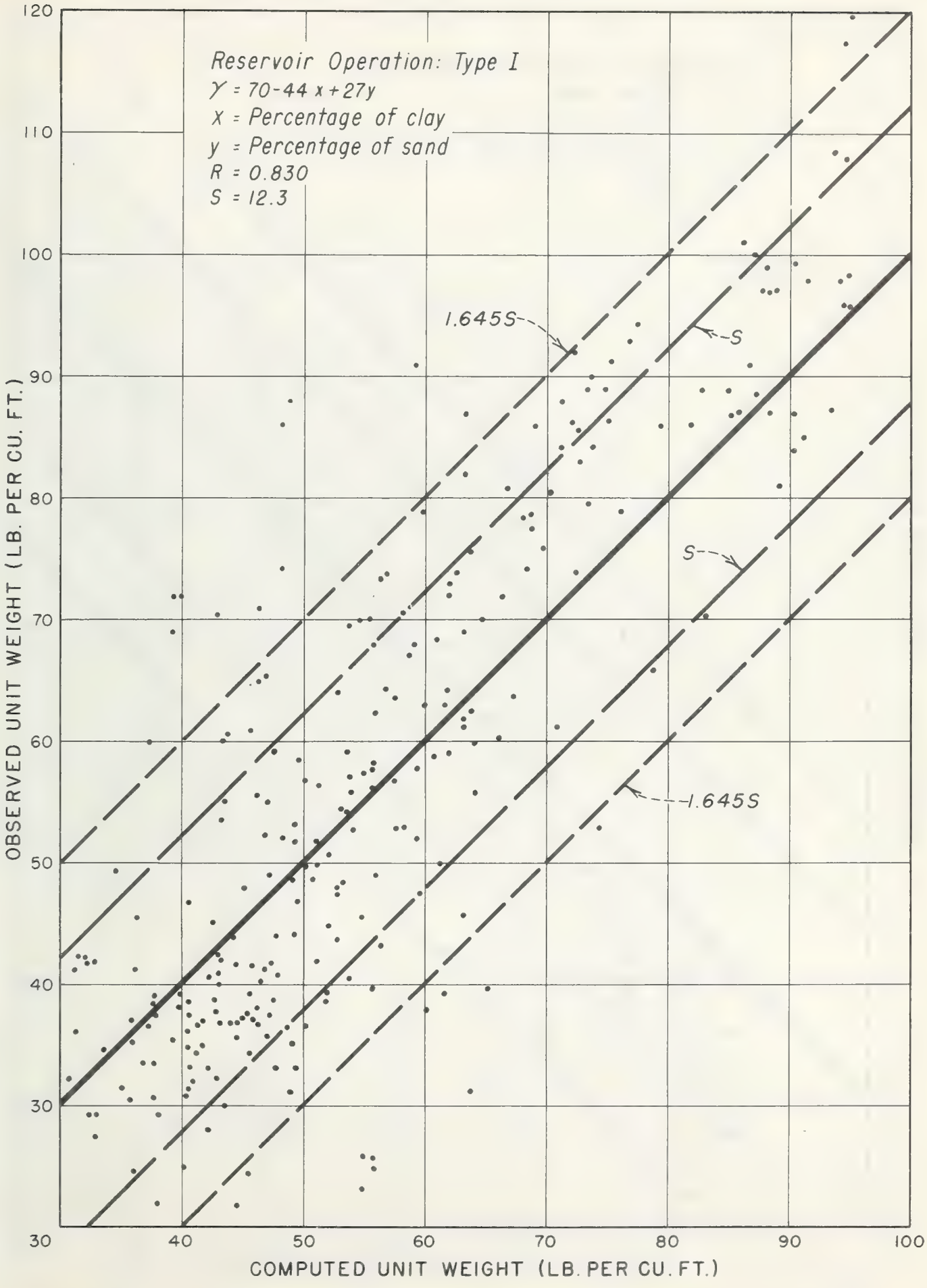


FIGURE 1. — Correlation of observed vs. computed unit weights. Reservoir operations: Type I.

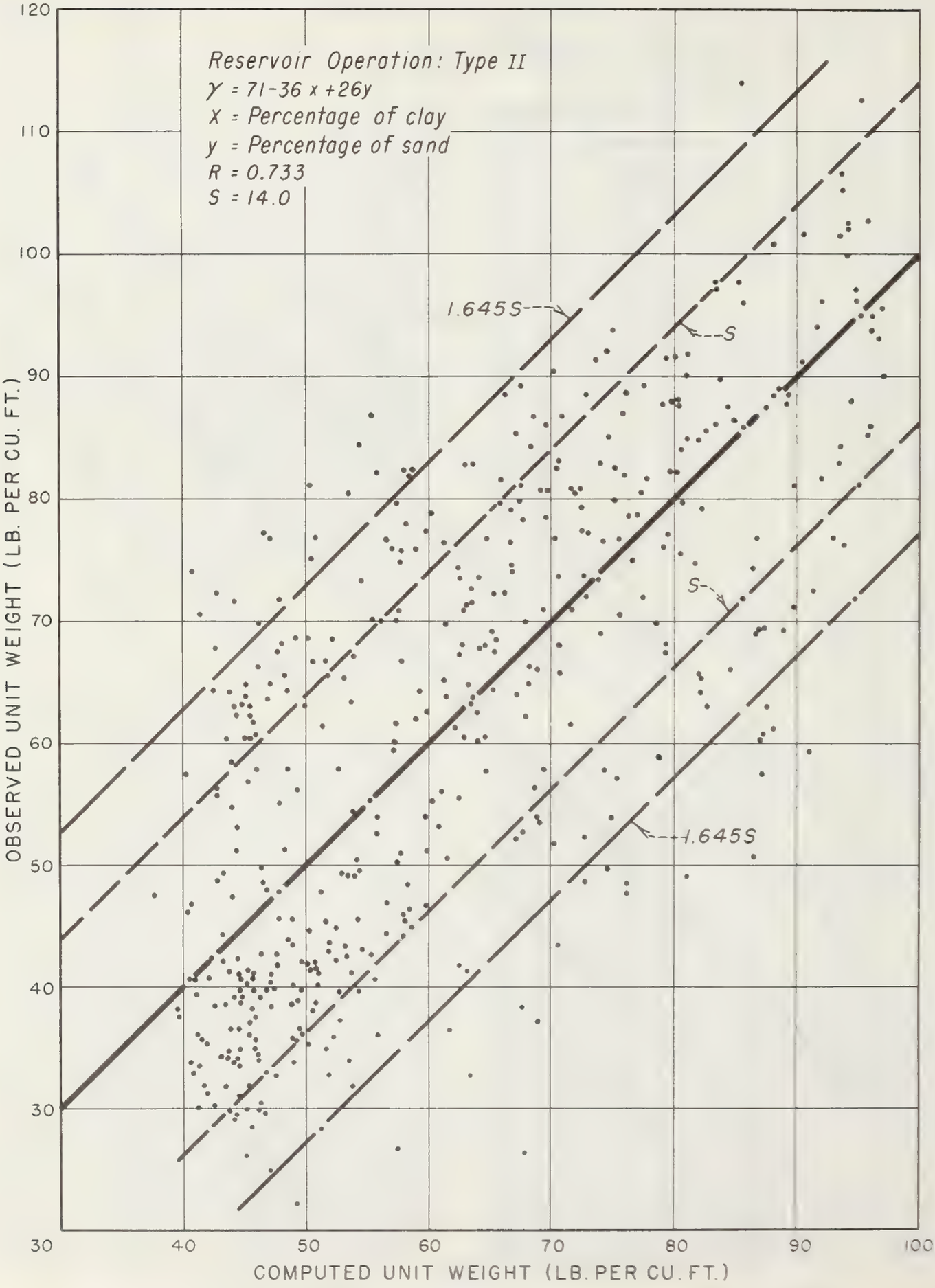


FIGURE 2. — Correlation of observed vs. computed unit weights. Reservoir operation: Type II.

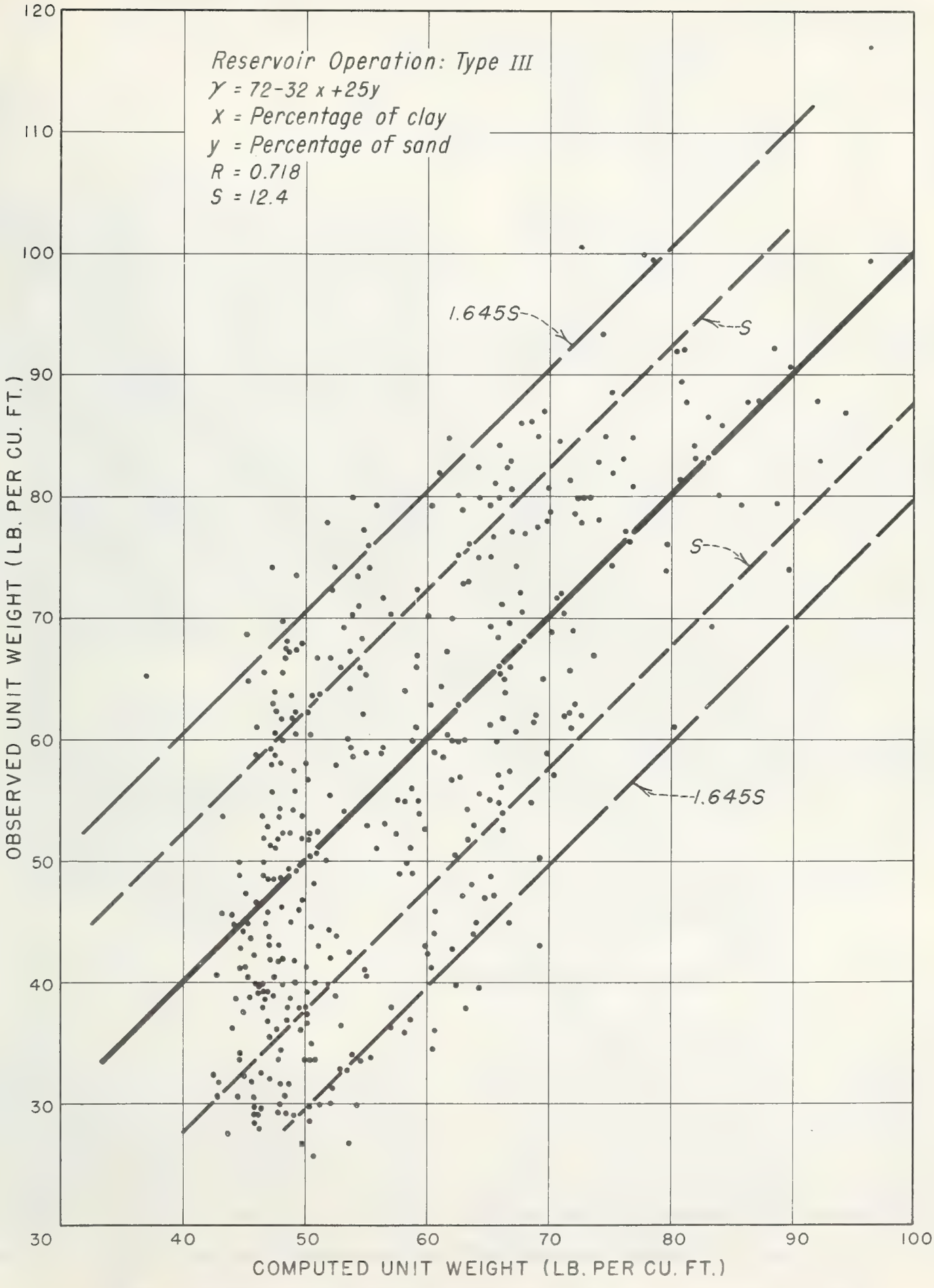


FIGURE 3. — Correlation of observed vs. computed unit weights. Reservoir operation: Type III.

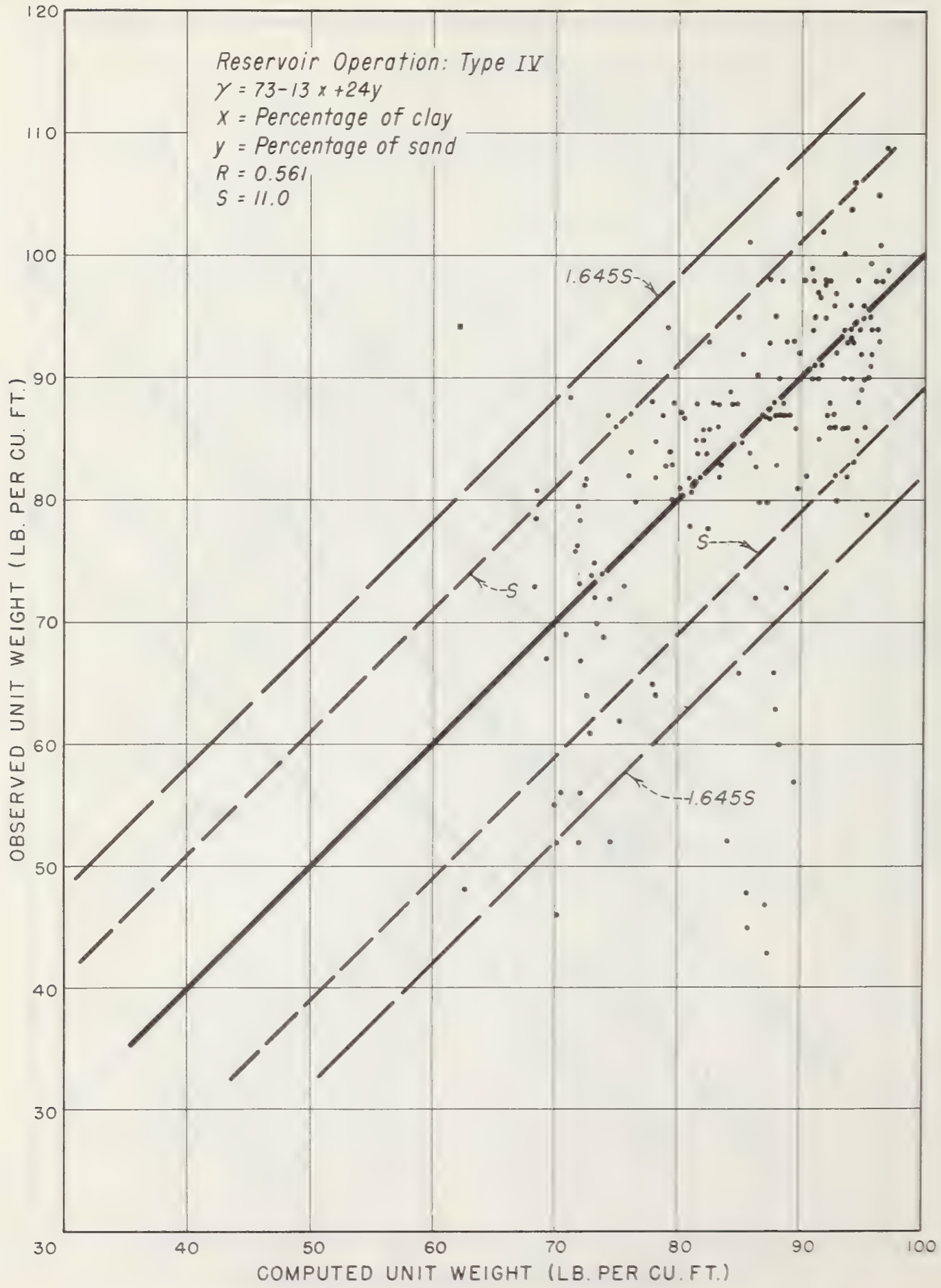


FIGURE 4. — Correlation of observed vs. computed unit weights. Reservoir operation: Type IV.

TABLE 3.—*Modified values of coefficients in equation 1 for reservoir types*

Type of reservoir operation	Observations	Values of coefficients in equation 1			R	S
		W_c	W_m	W_s		
	Number					*Lb./cu. ft.*
I	262	26	70	97	0.830	12.3
II	462	35	71	97	.733	14.0
III	405	40	72	97	.718	12.4
IV	187	60	73	97	.561	11.0

lines. One standard error of estimate would embrace about 68 percent of the residuals or differences between computed and observed values while about 90 percent would line within the range of 1.645 times the standard error.

The unit weight as determined from values of the above tabulation would be considered as the initial unit weight. No attempt has been made in this investigation to correct for the compaction of sediments as was done by Miller. This is a special phase and will be considered separately in a future study.

Sample Computation

An example of the computational procedure to determine the initial unit weight will now be given. The procedure simply entails the application of equation 1 for a given set of field data.

Given:

From the size analysis, it is found that the sediments are composed as follows:

Sediment type	*Percentage*
Clay	23
Silt	40
Sand	37

Reservoir operation: Type I.

Compute γ:

$$\gamma = W_c p_c + W_m p_m + W_s p_s$$
$$= (26)\ (0.23) + (70)\ (0.40) + (97)\ (0.37)$$
$$= 5.98 + 28.00 + 35.89 = 69.87 \text{ lb./cu. ft.}$$

This can be rounded to 70 pounds per cubic foot for practical use and for use as initial unit weight of sediment deposits.

Summary and Conclusions

It is concluded from this investigation that the unit weights can be computed on the basis of the regression equations derived from the analysis of the 1,316 samples compiled. This study showed better statistical results in deriving empirically the unit weight coefficients by equation 1 than by equation 2. Because of the greater number of samples analyzed, these coefficients would supersede those listed in Miller's [7] and the Lane and Koelzer[8] studies. It is suggested that the coefficients derived herein be used in future investigations where initial unit weight values need to be determined.

Future studies should be aimed at quantitatively determining the various factors affecting the unit weight of deposited sediments. This can be more readily accomplished by conducting sedimentation surveys of the type Heinemann [9] made for Sabetha Lake.

[7] See footnote 2.

[8] See footnote 1.

[9] HEINEMANN, H. G. VOLUME-WEIGHT OF RESERVOIR SEDIMENT. Amer. Soc. Civil Engin. Jour. Hydraulics Div. 88 (No. HY5): 181-197.

IMPROVED VOLUMETRIC SURVEY AND COMPUTATION PROCEDURES FOR SMALL RESERVOIRS

[Paper No. 83]

By H. G. HEINEMANN and V. I. DVORAK, *hydraulic engineers, Soil and Water Conservation Research Division, Agricultural Research Service*

Abstract

Hundreds of small reservoirs are surveyed every year to obtain information on storage capacity, sediment volumes, or for other purposes, at considerable time, effort, and expense for the associated field work and office computations. The final results from these surveys, however, are not always comparable or the data as useful as desired, because of poor procedures or techniques.

In a quest for the best survey and computation methods consistent with available resources, four small reservoirs were surveyed in great detail. Different surveys were imposed on these detailed bases and numerous computation techniques were used in the office calculations.

It was found that a field determination of the emergency spillway contour and a selected lower contour, together with data from sufficient ranges properly located (parallel where possible) provide adequate and dependable information. The integrimeter is a time-saving area-measuring tool that can be used to advantage.

Experience elsewhere and this particular study show that the stage-area curve method is the most direct, simple, accurate, and uniformly adaptable way to determine the capacity of a reservoir. Volumes of sediment deposits can best be obtained from capacity differences.

Introduction

Many small reservoirs are surveyed every year by Federal, State, and local agencies to obtain information on reservoir storage capacity, amount of deposited sediment, sediment distribution, and other factors. The need for good volumetric data in the design of water control and utilization projects has been so prominently emphasized in the past that it needs no further discussion here.

Considerable time, effort, and money are expended in the field and office work accompanying these reservoir surveys. However, the final results are not always as dependable as might be desired or as comparable with other reservoir data, because of the use of poor procedures. This situation should be improved.

This study was undertaken to determine the best procedures, consistent with the usually available resources, for determining the volume (remaining storage capacity) of a reservoir. Emphasis in this paper is on reservoir volumes rather than sediment volumes. Where sediment volumes are also desired, as pointed out in the paper, they can be obtained as the difference in remaining reservoir storage capacity at different dates. In the case of reservoirs without prior volume surveys, the measurements of sediment thicknesses can be made with a spud [1] or other acceptable device.

Volumetric Survey Procedure

The four reservoirs used as a basis for this study were selected because they were surveyed in great detail and they provided a variety of reservoir topography. They are relatively small, ranging in capacity from 29 to 70 acre-feet (figs. 1 to 4). The spacing of many of the cross sections near the dams is 25 feet, but spacings of 50 feet or more were used in the upstream parts

[1] EAKIN, H. M. (rev. by Brown, C. B.) SILTING OF RESERVOIRS. U.S. Dept. Agr. Tech. Bul. 524, 168 pp. 1939.

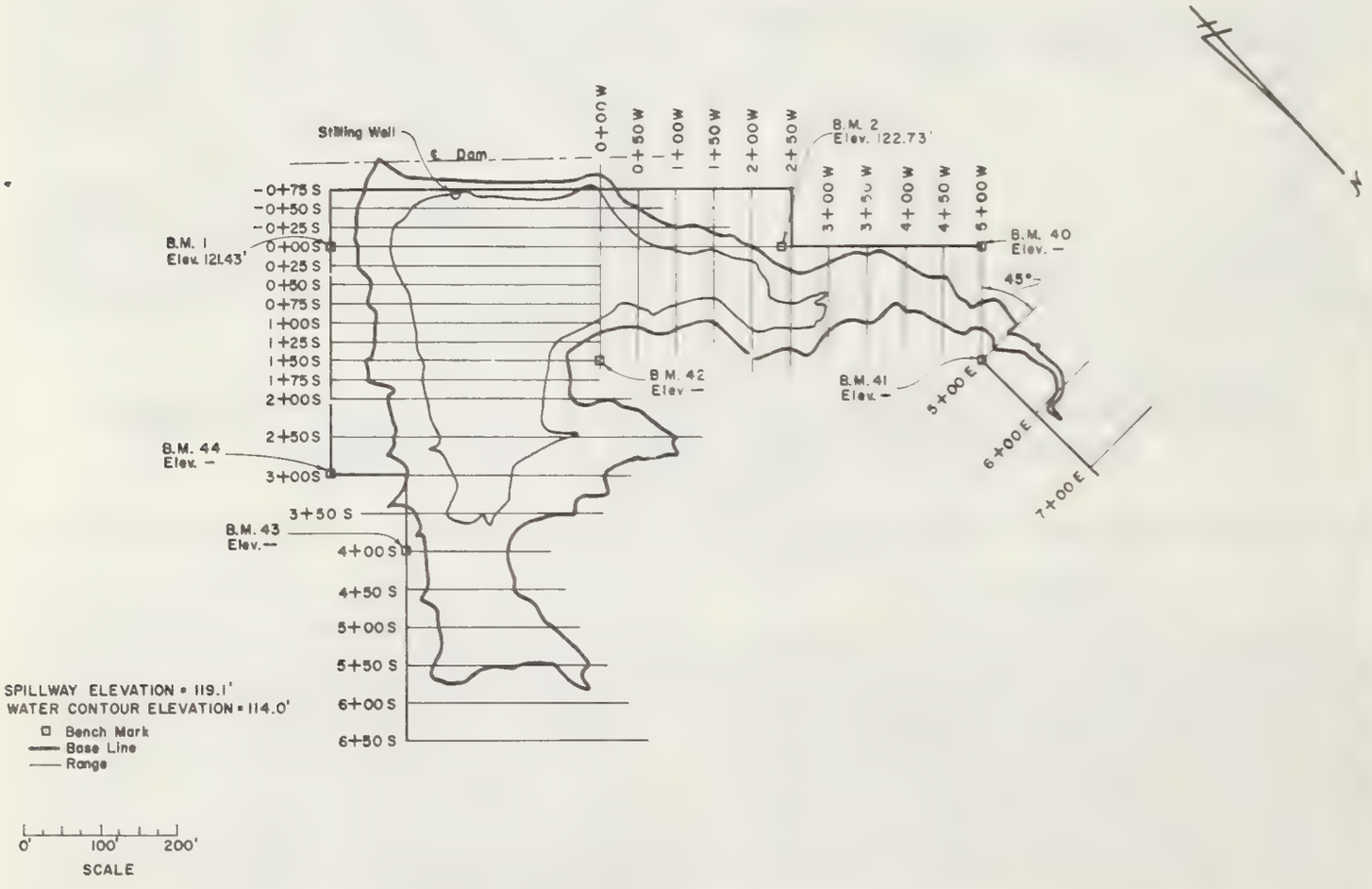


FIGURE 1. — Matt Hafner Reservoir, Newell, S. Dak., sedimentation survey, September 8, 1959.

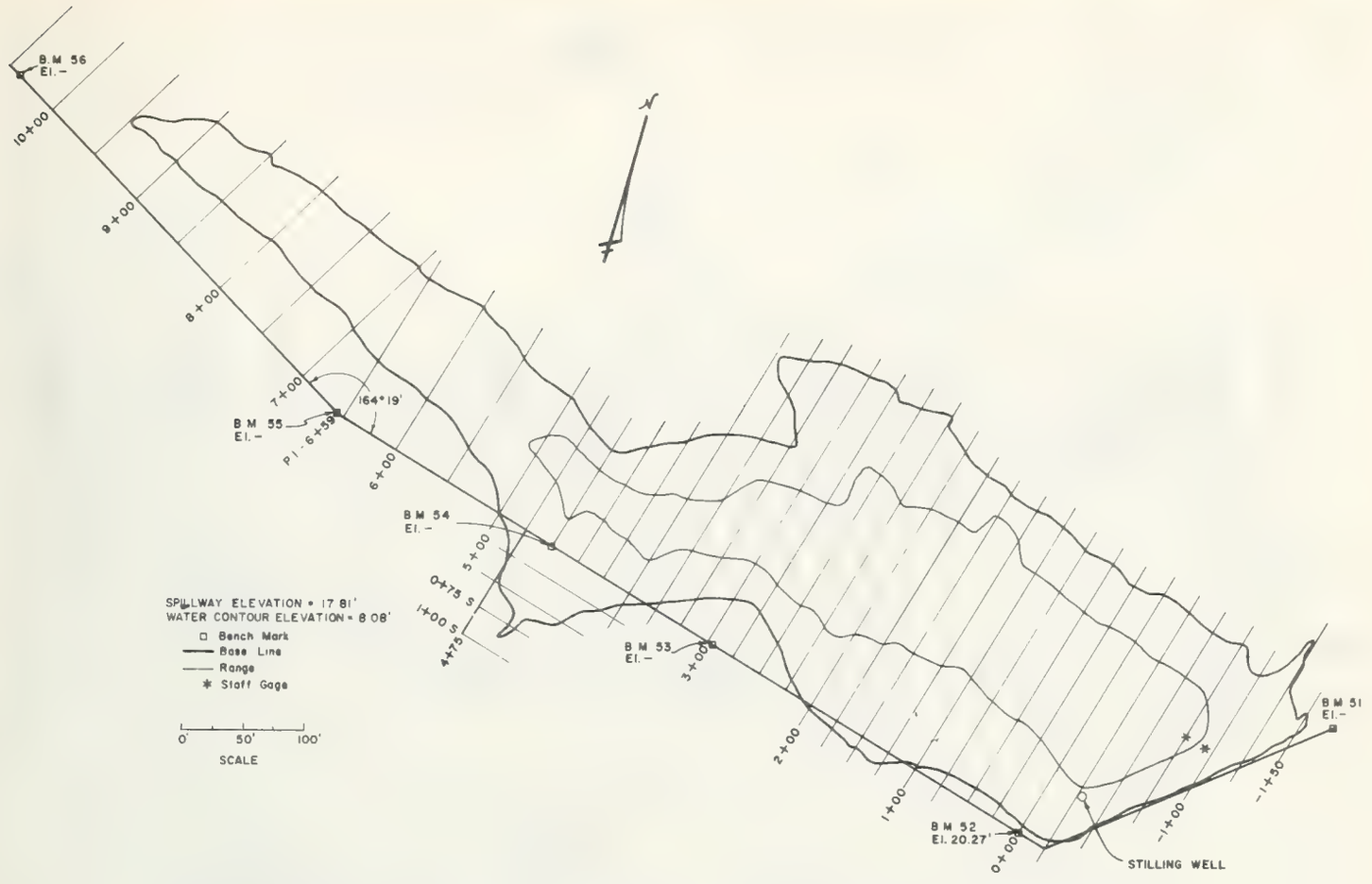


FIGURE 2. — Bertha Armour Reservoir, Newell, S. Dak., sedimentation survey, August 1958.

of the reservoirs. The survey method used on these four reservoirs proved very effective, and was used as a guide for the following suggested survey procedures.

Figure 5 is a drawing of a hypothetical reservoir showing many of the shapes encountered in the field. It will be used with the following discussion, which is applicable to the survey of a reservoir several years old and on which no survey base was established at the time the dam was completed.

Preliminary Field Work

Before actually starting field work, the engineer should study the topography in the surrounding area, especially that directly downstream from the dam. The slopes leading into the reservoir valley should be examined to determine if they are continuous or if a bench, terrace, or other irregularity exists. The presence of such topographic features will require consideration and adequate delineation during the fieldwork. Also, it is necessary to decide upon the method of computing the volume, since the field survey data required are partially dependent on the computational procedure.

Where possible, a measured base line should be established along one side of the reservoir, and this should generally be parallel to the main valley. Such a base line requires few permanent monuments and provides for greater accuracy in the survey itself.

Experience has shown that a reservoir contour map can be drawn adequately from a field determination of the upper spillway contour, a selected lower contour, and data from a sufficient number of ranges properly located. A range is simply a fixed line across a reservoir, along which an adequate number of elevations are determined to permit the establishment of the topography. The proper location of these ranges is facilitated if the plane table work locating contours is completed first.

Information as to the locations of the spillway contour (upper spillway — principal or emergency, depending on the type of structure) and another selected lower contour is desired, because these contours provide excellent horizontal control. The spillway contour should be mapped on a plane table sheet by using alidade and level equipment. This work proceeds quite rapidly if an engineer's level is used to insure that the rod is on the contour while the contour

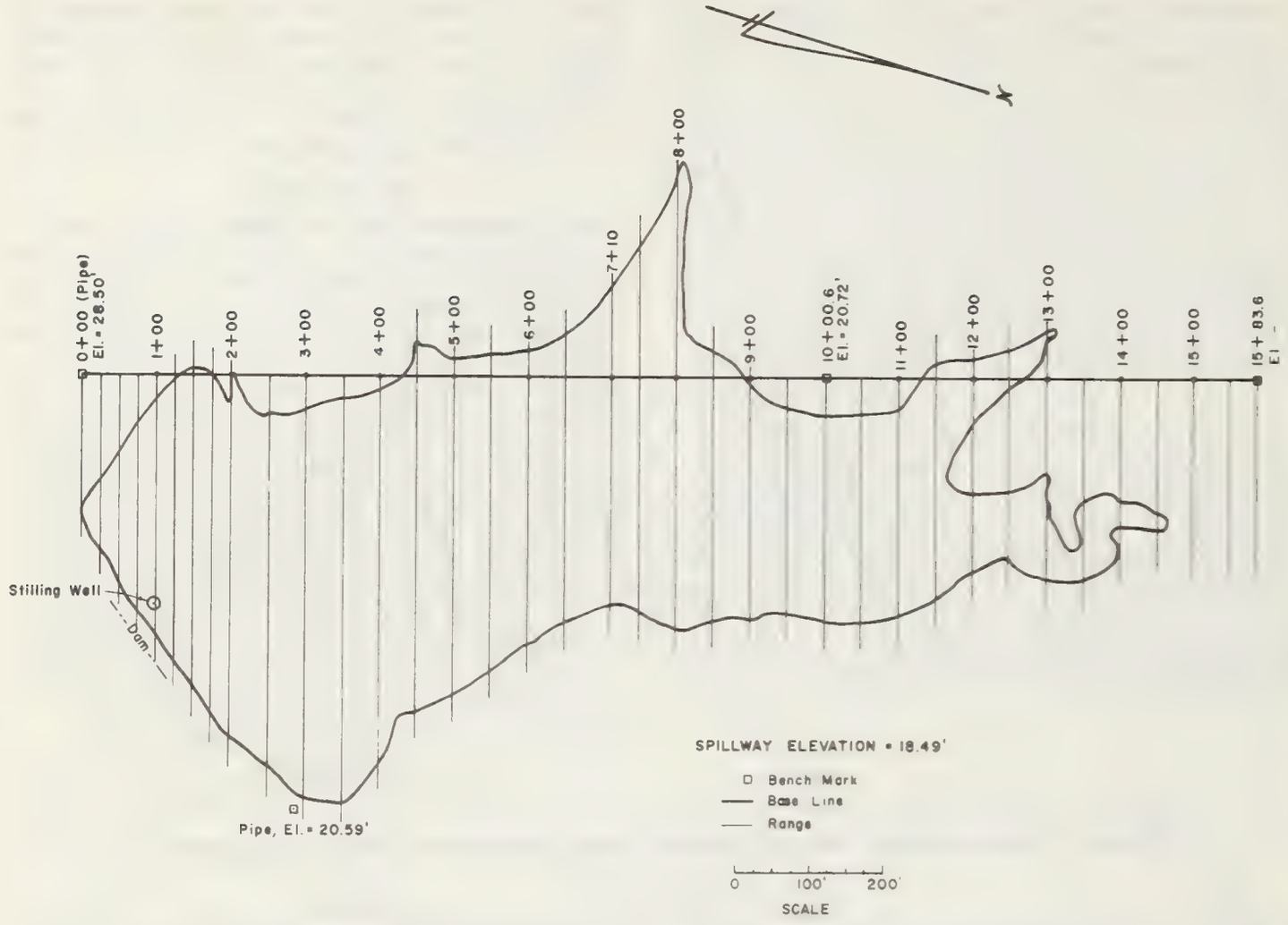


FIGURE 3. — Al Olsen Reservoir, Newell, S. Dak., sedimentation survey, July 1958.

is being located on the plane table sheet. A selected lower contour is desirable because it provides information on the changes in the land slopes between the ranges, and datum from it provides a strategic point on a stage-area curve. This selected lower contour is more valuable if it is located about one-third of the total reservoir depth down from the spillway. If the water stage is at this approximate elevation, the contour can be mapped rapidly, inasmuch as the level is not needed. If the reservoir is full, or nearly so, this lower contour is usually omitted.

Ranges

Ranges provide excellent vertical controls and should be located to yield maximum data for drawing contours. Ranges should be placed generally perpendicular to the valley and perpendicular to the base life, if possible. This greatly facilitates the computations. The ranges should be extended so that the maximum elevations are at least 3 feet above the highest spillway. It is important to have good range coverage so that the topography can be adequately portrayed.

If the exact topography of the upstream face of the dam is not known, at least one range should be located perpendicular to the axis of the dam to establish the presence or absence of a berm and also to locate the toe of the fill. If only a minimum number of ranges is used, one range must be located across the reservoir along the upstream toe of the dam.

Figure 5 shows a suggested range layout for a small reservoir. Most of the ranges are parallel to each other and perpendicular to the base line. Other ranges were placed in specific locations to facilitate obtaining special information.

Elevations along the ranges below the water level are determined by sounding with a sounding pole or bell, or other means. If the underwater topography is quite uniform, elevation determinations can be widely spaced. However, if the terrain is rough, the determinations should be numerous so that good topographic maps can be plotted. If a sounding bell is used, the graduations on the line should be checked

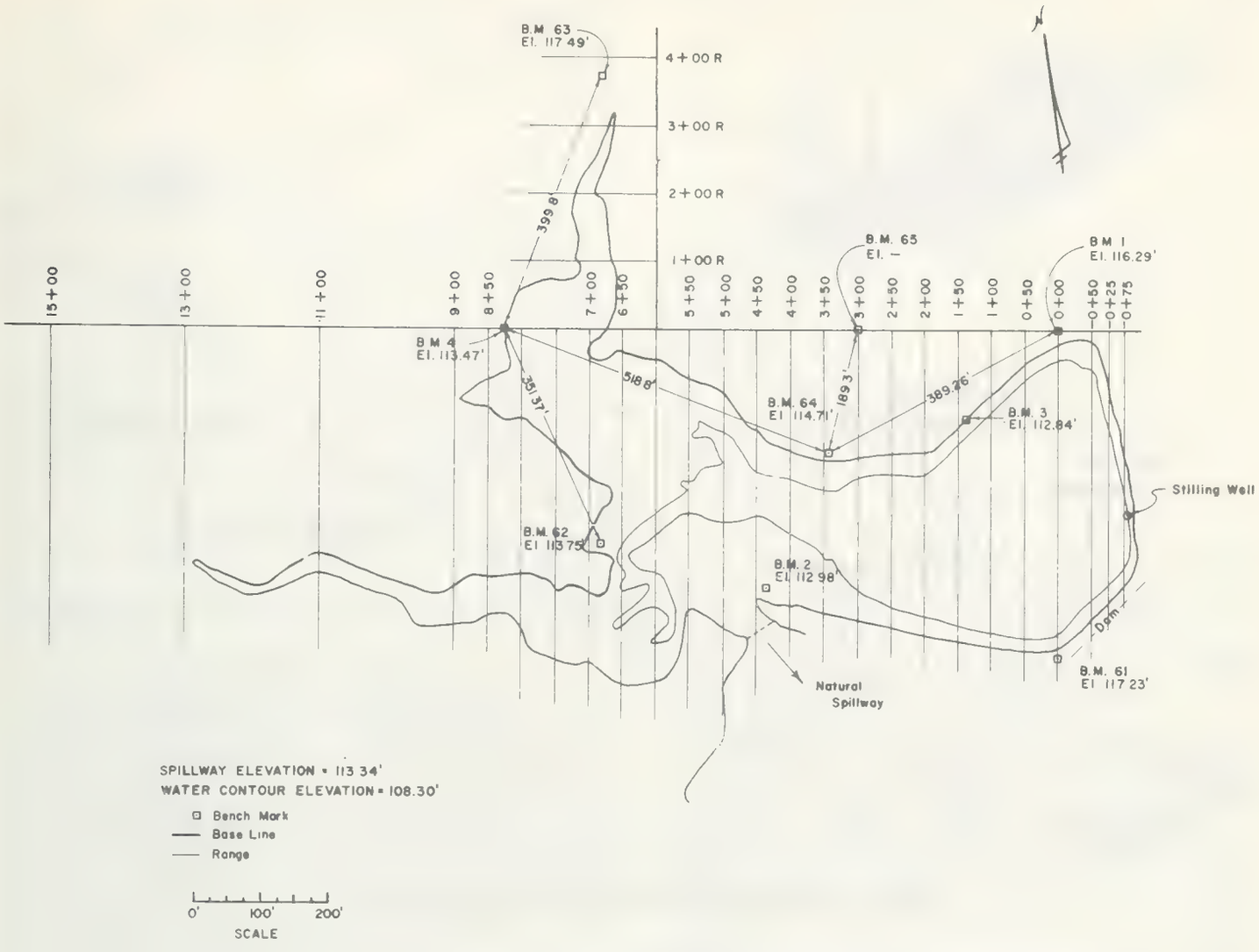


FIGURE 4. — Mitchell Reservoir, Newell, S. Dak., sedimentation survey, May 1960.

frequently for accuracy. Gottschalk[2] has described a system for marking the line. Elevations above the water level should be determined by standard level survey procedures.

If other data are to be obtained during the survey, such as the volume-weight of sediment, they should be obtained along the ranges. Accurate locations, both as to distance from range end and depth of sample, should be recorded.

In connection with all of the foregoing, it is more practical to obtain more than adequate, rather than insufficient data. It takes very little field time to obtain information along a few additional ranges, and such data may prove highly desirable during the computations.

Methods of Calculations

Numerous computational techniques were used to determine the reservoir capacities of the four reservoirs studied. These techniques were used with the following amounts of data: (1) the complete detailed reservoir surveys; (2) parts of the detailed surveys; and (3) imposed surveys developed from the detailed bases. The computational techniques can be grouped into two basic approaches—contour areas and cross-sectional areas.

Contour Area Methods

The contour area approach is dependent on contours to reflect actual topography adequately. Experience has shown that topographic maps with 2-foot contour intervals are adequate for the size of reservoirs represented by this study. This approach can be used with the following four methods: (1) Stage-area curve; (2) modified prismoidal; (3) Simpson's rule; and (4) average contour area.

Stage-area curve method

The stage-area curve method requires the plotting of a well-defined stage-area curve. If de-

[2] GOTTSCHALK, L. C. SOUNDING AND SPUDDING LINES. U.S. Soil Conservation Service Tech. Letter Sed-19, dated August 1, 1948.

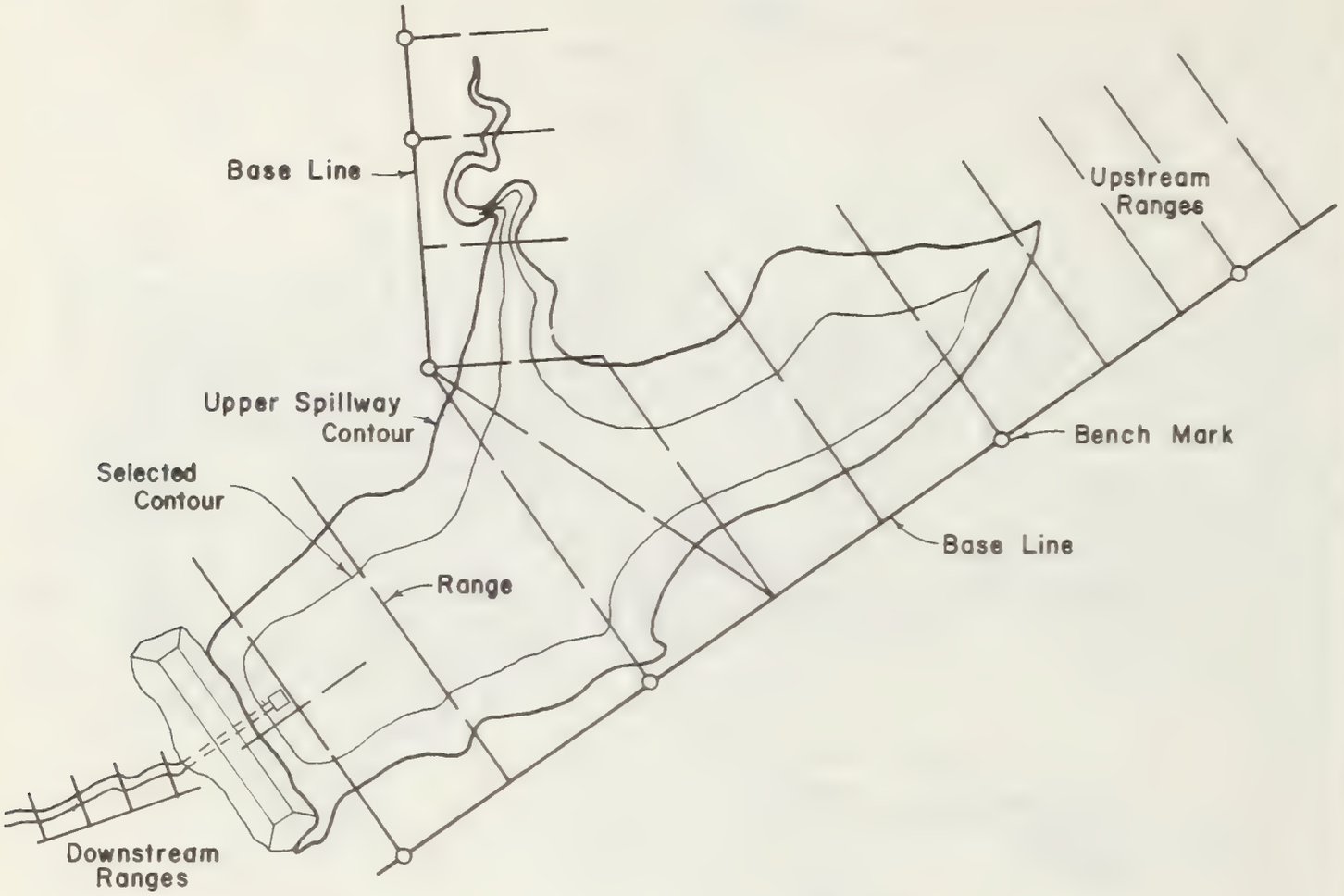


FIGURE 5. — Suggested range layout for a typical reservoir.

sired for Reservoir Sedimentation Data Summary forms,[3] such curves can be developed for various segments of the reservoir. The area between such a curve and corresponding selected elevations on the stage axis represents the reservoir capacity between the selected elevations. As an example, the capacity between elevations 112 and 114 (fig. 6) is represented by the shaded area. This can be planimetered and converted directly to capacity.

Modified prismoidal method

Figure 7 shows the equation used in the modified prismoidal method. The average of three areas is multiplied by the contour interval. The middle area is an approximation and an attempt to duplicate the midpoint of the segment on the stage-area curve. The accuracy of the results decreases with increased curvature in the stage-area curve.

In this method, the symbols represent the following:

V=capacity, in acre-feet;
L=contour interval, in feet;
A=area of lower contour, in acres;
B=area of higher contour, in acres.

Simpson's rule

Simpson's rule has never been mentioned in the reservoir survey literature, but it can be used to compute the capacity. A requirement is that the reservoir must be divided into an even number of segments. The general equation is

$$V=\tfrac{1}{3}\,h\,[\,A_o+A_n+4\,(A_1+A_3+\ldots+A_{n-1})+2\,(A_2+A_4+\ldots+A_{n-2})\,]$$

where

V=capacity, in acre-feet;
A=area of contour or cross section, in acres; and
h=interval spacing between contours or cross sections, in feet.

Average contour area method

This method consists of averaging the upper and lower contour areas and multiplying by the contour interval. The summation of such de-

[3] SUBCOMMITTEE ON SEDIMENTATION, INTER-AGENCY COMMITTEE ON WATER RESOURCES. SUMMARY OF RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1953. Sedimentation Bul. 6: Appendix. April 1957.

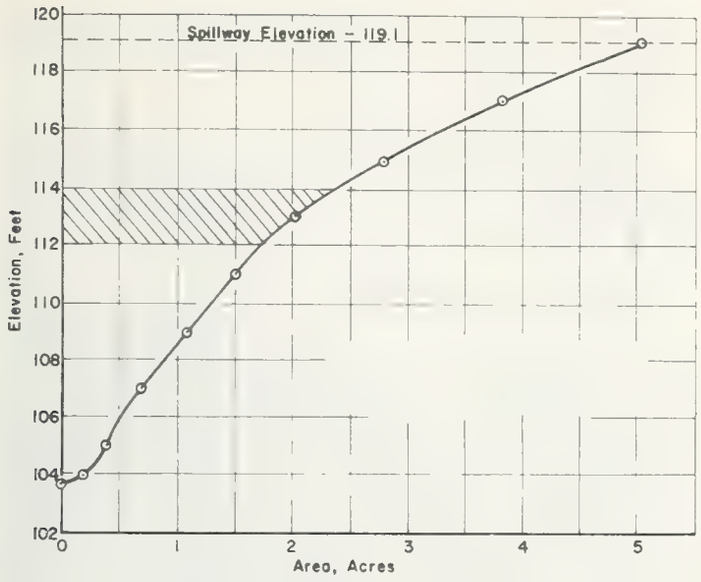


FIGURE 6. — Matt Hafner Reservoir stage-area curve.

terminations approximates the capacity of the reservoir.

Cross-Sectional Area Methods

The cross-sectional area approach depends primarily on cross-sectional areas (of ranges) in volumetric computations. The location of ranges, therefore, must be selected carefully to show the topography. This approach can be used with the following four methods: (1) Eakin's range end formula; (2) cross-sectional area vs. distance from dam — curve; (3) Simpson's rule, using the cross-sectional area method; and (4) average end area.

Eakin's range end formula

In this volumetric formula, both cross-sectional and surface areas are used, as explained by Eakin and Brown,[4] and also by Gottschalk.[5] It is based on the prismoidal formula and is shown in figure 8 with a segment illustration.

The symbols are:

V=capacity, in acre-feet;

A'=quadrilateral area — the area, in acres, of the quadrilateral formed by connecting the points of intersection of the ranges with crest contour;

A=lake area of the segment, in acres;

E=range cross-sectional area, in square feet;

W=width of the range at crest elevation, in feet;

h=the perpendicular distance, in feet, from one range to the shore line at the other range;

[4] See footnote 1.

[5] GOTTSCHALK, L. C. MEASUREMENT OF SEDIMENTATION IN SMALL RESERVOIRS. Amer. Soc. Civil Engin. Proc., Hydraul. Div. 77 (55) : 11 pp. 1951.

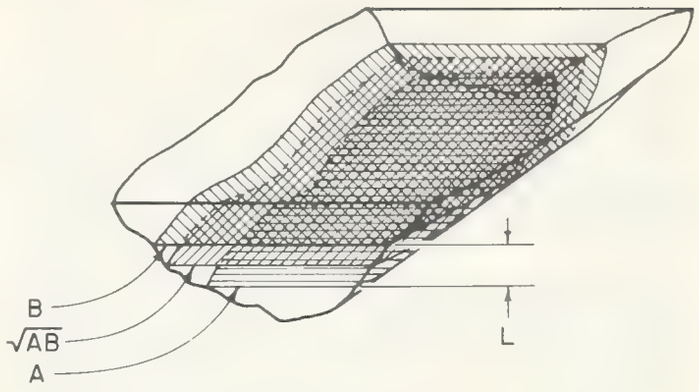


FIGURE 7. — Modified prismoidal formula.

h_3=the perpendicular distance, in feet, from the range on a tributary to the junction of the tributary with the main stream.

The subscripts shown in the figure identify these quantities with the respective ranges of the segment.

Reservoirs having irregular-shaped segments may require as many as five different variations of this formula to compute the total capacity. A standard form is available for uniform and orderly calculation of the basic formula. A template has been devised in conjunction with this form to speed the calculations and help prevent errors.

Cross-sectional area vs. distance from dam— curve method

On a graph showing range cross-sectional areas plotted versus distances from the dam, the capacity is represented by the area between the curve and the "distance from the dam" axes. The accuracy is dependent on the ade-

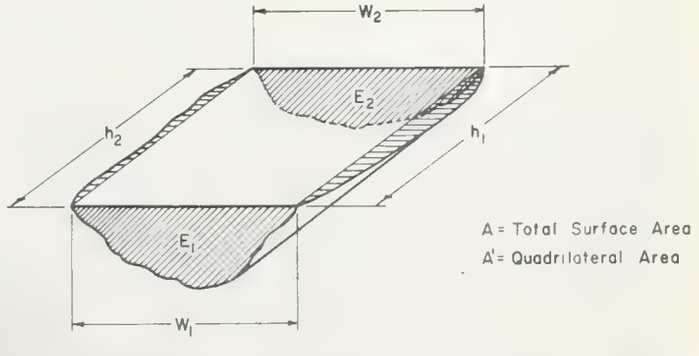


FIGURE 8. — Eakin's range formula.

quacy of the defined curve. Figure 9 shows such a curve.

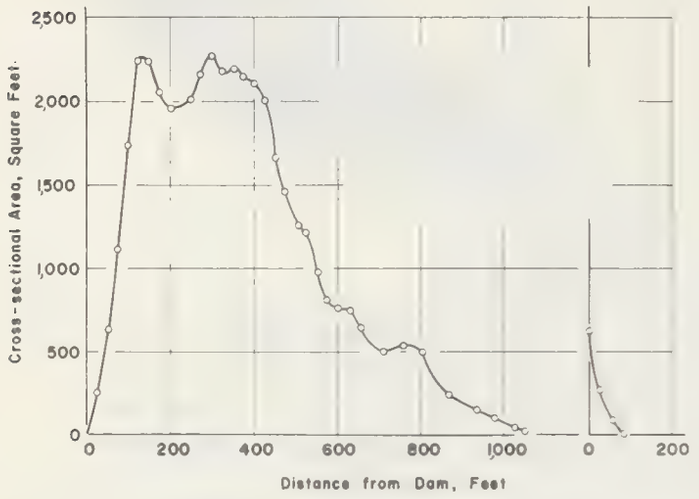


FIGURE 9. — Bertha Armour Reservoir cross-sectional area versus distance from dam.

Simpson's rule

The use of Simpson's rule, with the cross-sectional area approach, was made possible by the many evenly spaced parallel ranges. In several instances, it was necessary to use another cross-sectional area method to compute the volume of the final odd segment.

Average end area method

The average end area method requires a determination of the average of the cross-sectional areas. This average is then multiplied by the total length of the reservoir to obtain the capacity.

Sediment Volume

The easiest and most practical method of computing the volume of sediment deposited in a reservoir between certain survey dates is by determining the difference in capacities on the two dates. In the case of a reservoir without satisfactory prior volume surveys, sediment thicknesses can be determined with a spud or other suitable device and the necessary information obtained for establishing the original contours or cross-sectional areas of the reservoir. In this latter case, the sediment volume is the difference between the original reservoir capacity and that remaining at the date of survey.

Short Cuts

Time-saving devices and procedures were used whenever possible. All measurements on the stage-area curve and contour area measure-

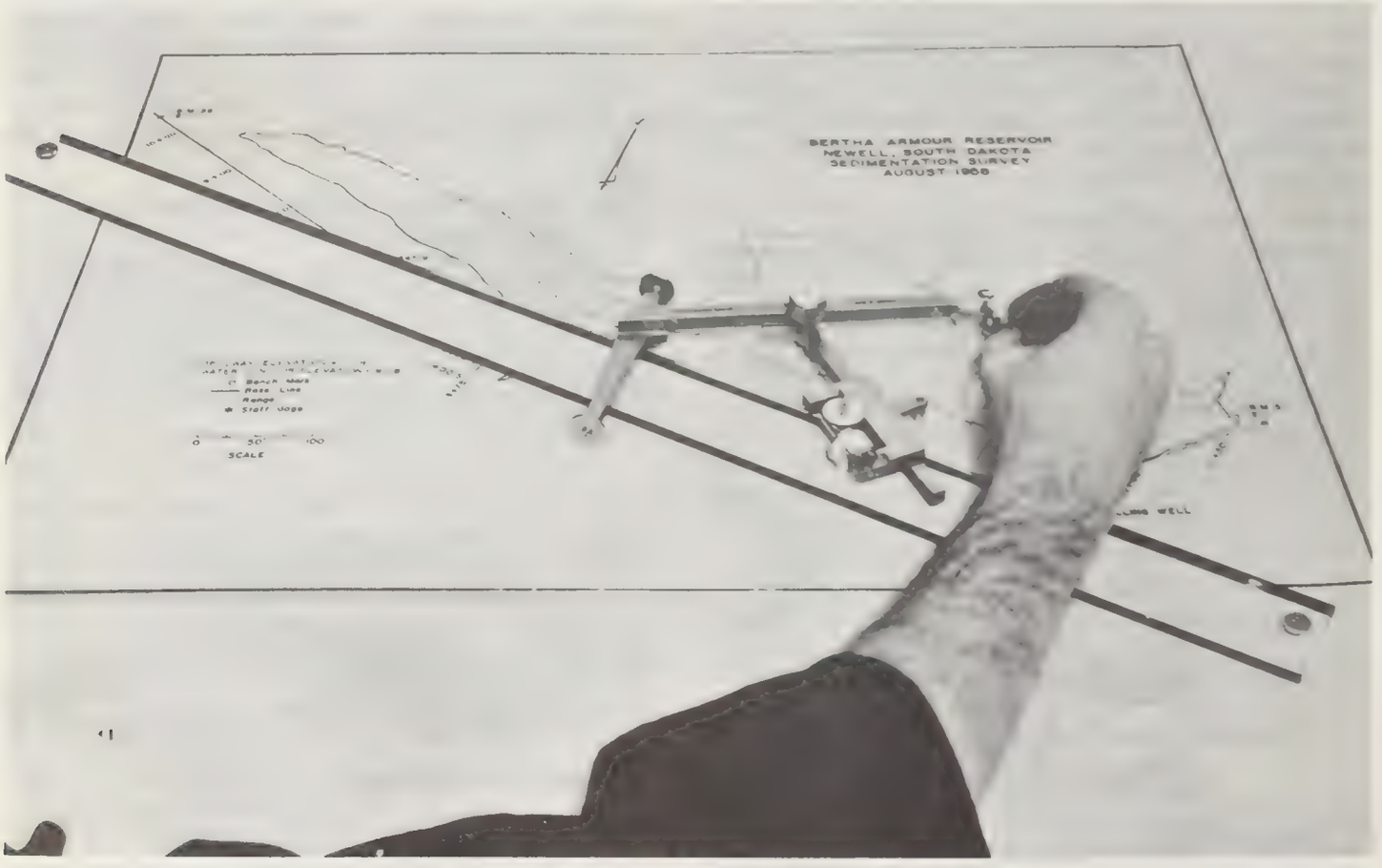


FIGURE 10. — Integrimeter.

ments when the ranges were parallel to each other were made with the integrimeter. Figure 10 shows this instrument. Its use saves a great deal of time. It determines the integral value from point to point on a curve while tracing the curve; it determines directly the area below any portion of a traced curve. It can also be used as a planimeter. It has another advantage in that the measuring wheel runs on a special surface at all times, minimizing differential wheel slippage.

A short-cut method for determining cross-sectional areas directly from reduced field notes has been developed by Mr. Dvorak. This method is a programming of a general trapezoidal formula for determining cross-sectional areas on an electric calculator having a single keyboard. This method saves from one-third to two-thirds the normal time required to determine such areas. Furthermore, a check is made of all entries simultaneously with the computations. Figure 11 shows the tabulation forms used with Dvorak's method. It shows the simplicity of the tabulation involved in the computation and its check for each of Dvorak's two methods.

SUMMARY FORM - CROSS SECTIONAL AREAS
COMPUTED BY THE ELEVATION METHOD

Reservoir _____
State _____
Survey Date _____

Computer _____ Date _____

Range or Cross Section No. _____

Calculation Check: Σ(Distance + Elevation) = _____

(Distance_____) x (Elev. Spwy. _____) = _____

minus Double Area (_____) ½ = _____

Cross Sectional Area (Ft.^{2}) = _____

SUMMARY FORM - CROSS SECTIONAL AREAS
COMPUTED BY THE DEPTH METHOD

Reservoir _____
State _____
Survey Date _____

Computer _____ Date _____

Range or Cross Section No. _____

Calculation Check: Σ(Distance + Depth) = _____

Double Area = _____

Cross Sectional Area (Ft.^{2}) = _____

FIGURE 11. — Illustration of form used in computing cross-sectional areas in the elevation and depth methods (after Dvorak's shortcuts).

Results

The field survey and the office computations are used together to obtain reservoir capacity. The best type of computation procedure is of little value without an adequate field survey. The fieldwork should be oriented with the computation method so that the essential information is always obtained.

The results of the various computations applied to the data for the small reservoirs are shown in tables 1 to 4. A study of these tables

TABLE 1.—*Summary of capacity computations—Al Olsen Reservoir*

Methods of computation	Contour interval	Type of survey	Remaining capacity
	Feet		*Acre-feet*
Stage-area	2	34 parallel ranges...	68.06
	4	34 parallel ranges...	68.01
	2	4 parallel ranges, 12-foot contour	68.56
	2	7 skewed ranges, 12-foot contour	69.16
Modified prismoidal	2	34 parallel ranges...	67.69
	4	34 parallel ranges...	67.36
	2	4 parallel ranges, 12-foot contour	68.74
	2	7 skewed ranges, 12-foot contour	69.33
Eakin's range formula.	.....	34 parallel ranges...	68.23
	.....	4 parallel ranges...	72.43
	.....	7 skewed ranges...	64.07
Simpson's Rule (range cross-sectional area).	2	34 parallel ranges...	67.81
	.....	34 parallel ranges...	67.69
	.....	29 parallel ranges...	67.54
Average contour area	2	34 parallel ranges...	68.08
	4	34 parallel ranges...	68.80
	2	4 parallel ranges, 12-foot contour	69.11
	4	4 parallel ranges, 12-foot contour	70.49
	2	7 skewed ranges, 12-foot contour	69.71
	4	7 skewed ranges, 12-foot contour	70.93
Cross-sectional area versus distance from the dam.	.....	34 parallel ranges...	68.34

reveals that when all the available survey data (all information from ranges spaced 25 or 50 feet) were used with each of the methods of computation, the capacities were almost identical. With a great deal of data, any method is adequate. Some methods are a little cruder than others, and this is generally shown in the results.

The stage-area method result was considered to be the most accurate and was used as a basis for other comparisons. This method is more direct than others and does not use approximation formulas.

TABLE 2.—*Summary of capacity computations—Bertha Armour Reservoir*

Methods of computation	Contour interval	Type of survey	Remaining capacity
	Feet		*Acre-feet*
Stage-area	2	39 parallel ranges	26.14
	2	6 parallel ranges, 8.08-ft. contour	25.96
Modified prismoidal	2	39 parallel ranges	26.06
	2	6 parallel ranges, 8.08-ft. contour	25.67
Eakin's range formula.	.....	39 parallel ranges	26.08
	.....	6 parallel ranges	24.15
Simpson's Rule (range cross-sectional area).	2	39 parallel ranges	26.18
	.....	39 parallel ranges	26.07
Average contour area.	2	39 parallel ranges	26.14
	2	6 parallel ranges, 8.08-ft. contour	25.78
Average end area	.....	39 parallel ranges	26.19
Cross-sectional area versus distance from the dam.	.....	39 parallel ranges	26.20

TABLE 3.—*Summary of capacity computations—Matt Hafner Reservoir*

Methods of computation	Contour interval	Type of survey	Remaining capacity
	Feet		*Acre-feet*
Stage-area	2	34 parallel ranges	29.66
	2	9 parallel ranges 14-ft. contour	29.58
	4	9 parallel ranges, 14-ft. contour	29.81
	2	7 skewed ranges, 14-ft. contour	29.73
	4	7 skewed ranges, 14-ft. contour	29.76
Modified prismoidal	2	34 parallel ranges	29.63
	2	9 parallel ranges, 14-ft. contour	29.60
	2	7 skewed ranges, 14-ft. contour	29.09
Eakin's range formula.	.....	34 parallel ranges	29.63
	.....	9 parallel ranges	30.74
	.....	7 skewed ranges	30.26
Simpson's Rule (range cross-sectional area).	.....	34 parallel ranges	29.70
Average contour area.	2	9 parallel ranges, 14-ft. contour	29.74
	2	7 skewed ranges, 14-ft. contour	29.71

When a study was made of the results obtained by using the data from a small number of parallel ranges, the stage-area method capacities more nearly approached the volumes determined by using all the data. This was followed by the results from other methods based on contour areas and then by those involving cross-sectional areas. Normally, more than four ranges are needed in determining the remaining capacities. Figure 12 shows the few selected parallel ranges and the skewed ranges that were used on the Al Olsen Reservoir for comparative purposes.

TABLE 4.—*Summary of capacity computations—Mitchell Reservoir*

Methods of computation	Contour interval	Type of Survey	Remaining capacity
	Feet		*Acre-feet*
Stage-area	2	26 parallel ranges	38.31
Modified prismoidal	2	26 parallel ranges	38.41
Eakin's range formula.	.....	26 parallel ranges	38.19
Simpson's Rule (range cross-sectional area).	.....	26 parallel ranges	38.59
Average contour area.	2	26 parallel ranges	38.71
Average end area	.....	26 parallel ranges	38.24

When only the data along a few skewed ranges were used in the computations, the stage-area method again provided the best results. Figure 12 shows a suggested range layout for a reservoir on which parallel ranges are not practical. Note that they were so selected that, in conjunction with the selected contours, they furnish a maximum of topographic data.

Different individuals surveying this reservoir might select different range layouts and thus obtain different data for the computations. These would result in different reservoir capacity values. They might or might not be as good as those shown in table 1. Not all possible computation methods were tried with the data from the few ranges shown in figure 12 because some were no longer applicable.

A selected lower contour was used to provide additional data on topography. This information was useful with the contour approach but such supplemental information cannot be inserted into the cross-sectional area approach.

Accurate records of the time required for each computation method were not maintained. However, it is believed that the desired remaining capacity, stage-storage and stage-area curves, and other information useful on the Reservoir Sedimentation Data Summary forms, can be obtained in the shortest time with the stage-area method. It is quicker than the other methods using the contour approach because formula calculations are not involved. Experience with the various methods also indicates that the stage-area method is faster than the methods employing cross-sectional areas. The stage-area method is also more uniformly adaptable than most of the other methods. It does not require separate formulas for various segments of the lake, nor does it require an even number of con-

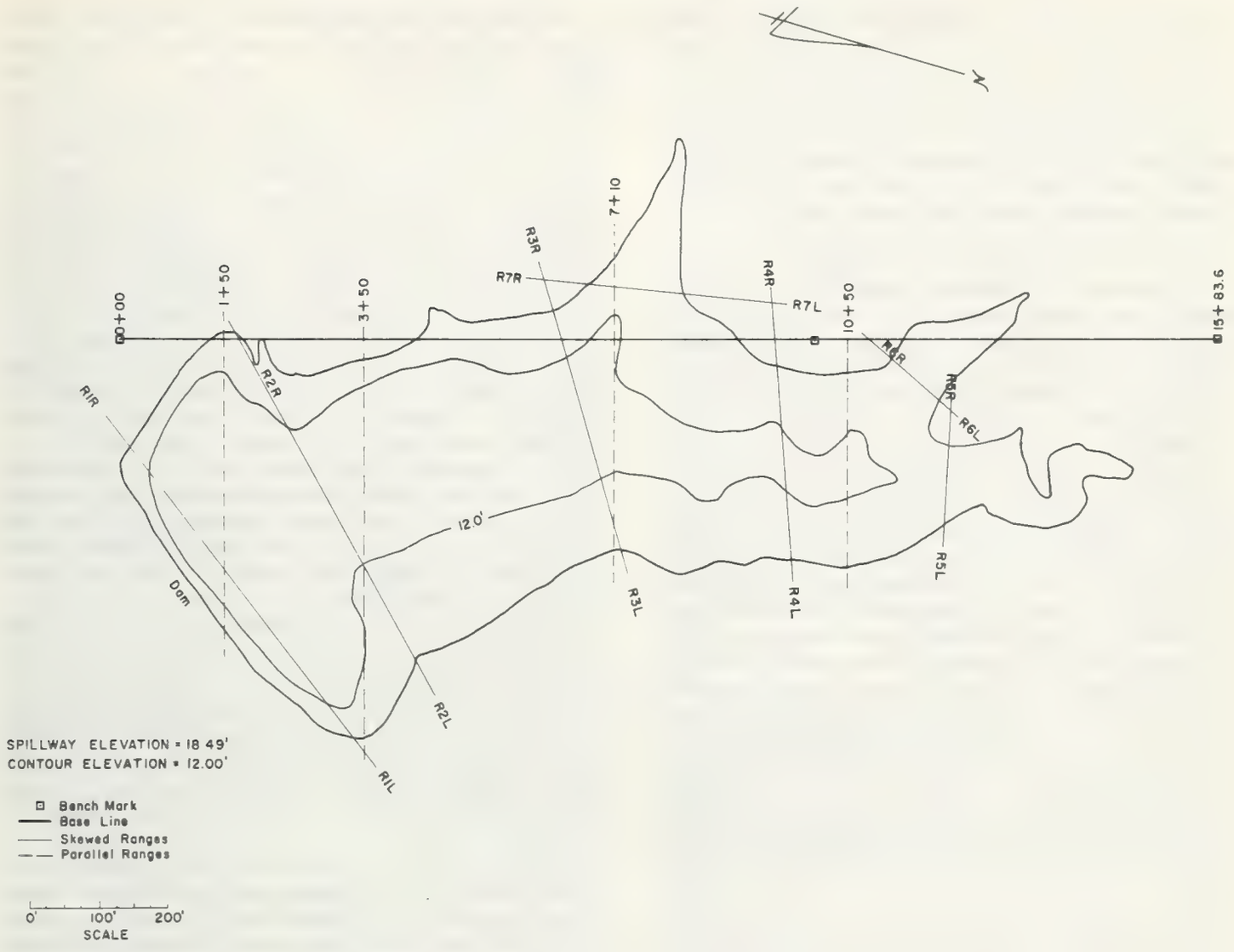


FIGURE 12. — Imposed survey, Al Olsen Reservoir.

tour areas or even spacing of ranges or contours.

Discussion and Conclusions

Volumetric Survey

The suggested survey procedure discussed previously was developed after considerable fieldwork experience in determining capacities. These experiences have shown that a base line along the side saves time. It permits the plane table work to proceed with measured distances always available for verification. With such a base line and ranges perpendicular to it, ranges can be located by distances along this line, and no range end monuments need be established.

The spillway and selected lower contours provide substantial information for drawing the other contours. The latter is helpful whatever its elevation, but the added information is minimized if this contour is relatively close to the spillway contour or near the bottom of the reservoir.

There are instances where the base line along the side and determining the location of the lower contour are not practical. In these cases, select the best base line available and locate the range ends on the plane table sheet by distance measurements and by triangulation. Where the location of a selected lower contour is not practical in the field, the ranges should be spaced closer so that the topography can be determined adequately.

It is important to acquire adequate data in the field. The location of an extra range or two and the obtaining of information along such ranges take little time. The field surveys should be oriented toward the capacity calculation method.

The results tabulated in the preceding tables show that the detailed surveys actually made of

the four reservoirs were not necessary. A capacity value that is practically the same as that computed from the detailed survey was obtained with considerably fewer ranges carefully located, the upper spillway contour, and a selected lower contour. The suggested type of survey requires considerably less field and office time, fewer permanent markers, and less funds.

Methods of Computation

Experience elsewhere and this particular study show that the stage-area curve method is the most direct, simple, accurate, and uniformly adaptable way to determine the capacity of a reservoir. This method provides more easily the remaining capacity, the stage-storage curve, and other information.

This method also exploits more fully the advantage that all contour area methods have over the cross-sectional area methods; namely, that the stage-area curve can be plotted easily for the entire reservoir or segments of it. When the work is done by segments, information is obtained in regard to capacity changes with increased distance from the dam.

Experience in the association of reservoir topography with corresponding stage-area curves can be used in determining what fieldwork is needed to provide adequate data for capacity determinations. As an example, if experience in particular topographic areas indicates that stage-area curves are likely to be straight lines, minimum field survey data will probably be adequate.

The modified prismoidal method requires more calculations than the stage-area curve method; yet, it is not as accurate if the stage-area curve is irregular. Furthermore, judgment is involved in the calculations and there is added opportunity for errors.

The Simpson's rule requires an odd number of equally spaced contours or ranges. If this cannot be obtained easily, it often necessitates the use of another method to compute the capacity of the extra segment to complete the total capacity determination. The formula provides a close approximation of the actual capacity, but its use is limited.

The average contour area method is simple, but an average of two contour areas does not always approach the correct value, especially if there is a large difference in the relative sizes of two adjoining areas. Additional calculations are also required, which increases the possibilities of errors.

The capacity determination by Eakin's range formula requires a large amount of calculations using approximation equations. The equations depend on segment shape and location. Different ones are required for (1) the first segment next to the dam; (2) triangular-shaped segments caused by side tributaries; (3) long, meandering segments; (4) relatively straight segments; and (5) end segments. There are several equations for some segments, and the most appropriate one must be selected on the basis of judgment. Furthermore, the location of ranges is more critical when used with Eakin's method than with the stage-area curve method. This observation is confirmed from range layouts shown in figures 3 and 12 and the tabulations shown in table 1. Supplemental information cannot be readily utilized in this method even though it is easily obtained.

The cross-sectional area versus the distance from the dam method requires that the ranges be located approximately perpendicular to the valley and parallel to each other. There also must be a sufficient number of ranges so that the curve is well defined.

The average end area method applied to range cross-sectional areas is simple, but it also has the same disadvantage as the average contour area method. An average of two cross-sectional areas does not always approach the real one, especially if there are appreciable differences in the areas. Furthermore, this method cannot provide stage-area or stage-capacity information.

Whenever the ranges are approximately perpendicular to the valley and parallel to each other, a plotting of the cross-sectional areas of ranges versus the distance of the ranges from the dam can reveal shortcomings in the range layout. If this plotting shows irregularities that are not adequately supported by plotted points, then additional ranges need to be established at those locations to provide the missing data. When sufficient points are available so that the curve is accurately defined, the range layout and the associated volumetric determination will be adequate.

Acknowledgments

Larry Reynolds, hydraulic engineer; Elmer Mandrup, engineering technician; and Charles Hruby, formerly an employe, Agricultural Research Service, did most of the detailed office work in the experimentations with the different methods of computation. Other fellow Agricultural Research Service workers aided substantially in the field surveys and office computations and drafting. A. L. Sharp, supervisory hydrologist, Agricultural Research Service, was very helpful with his review comments.

DETERMINATION OF RESERVOIR DEPOSITS BY RECONNAISSANCE METHODS

[Paper No. 84]

By GLENN H. CASTLE, *supervisory hydraulic engineer, U.S. Army Engineer District, Sacramento*

Synopsis

Sedimentation estimates for many large reservoirs, such as those in the Sierra Nevada Range in California, indicate filling rates that will not significantly affect their operation for 50 to 100 years, unless the sedimentation is concentrated to an unforeseen degree at some critical point within the reservoir where it might interfere with the behavior of special works such as diversion intakes, bridge crossings, or nearby roads. In such reservoirs, occasional resurveys may be required, mainly to detect gross errors in estimated sedimentation rates or unusual or unexpected concentration of sediment at critical points, rather than for detailed information on exact sedimentation rates and information on specific gravity and grain size of sediments. Experience indicates that occasional aerial and/or ground reconnaissance of such reservoirs may be adequate for these purposes and can be accomplished for only a few hundred dollars rather than the thousands of dollars needed for a standard hydrographic or land resurvey of the established range system. When a reconnaissance indicates sufficient sedimentation to justify a more complete survey, information obtained by such reconnaissance can usually be used to reduce the scope of the required survey sufficiently to more than recover the cost of the reconnaissance. More use of reconnaissance methods is recommended.

Determination of Reservoir Deposits by Reconnaissance Methods

Current knowledge of the amount, type, and location of sediment deposits in a reservoir may be of major importance where sediment rates are relatively high and available space is limited, but deposits may be of only minor interest if rates are relatively low and available space is adequately large. A study of estimated sediment rates and of the planned size and method of operation of the reservoir will usually indicate the probable severity of the sediment problems and the frequency of required resurveys.

Most major streams rising in the Sierra Nevada Range of California have large parts of their watershed above the winter snowline, and a major portion of their annual runoff occurs during the spring months as a result of melting snow. Snowmelt runoff rates are moderate and result in relatively low sediment production rates, with most of the transported material consisting of sand and heavy silt moving as bedload. Major rainstorms also occur on these streams and result in high flows of short duration and low volume. These short duration flows have higher sediment production and transport rates, both as bedload of sand and gravel and as suspended load of silt and clay. The combination of large streamflow volume having relatively low sediment rates and small streamflow volume having higher sediment rates is such that the combined total annual sediment production and transport rates are relatively low on most of these streams. Although predicted annual sediment rates are low, it has been considered desirable to install permanently monumented sediment ranges in many of these reservoirs in order to measure total sediment deposition during their long anticipated lives. Where such ranges have been installed, adequate information on progressive sedimentation can often be obtained by only infrequent resurvey of the entire range system if this information is supplemented by more frequent reconnaissance surveys.

In the Sierra Nevada reservoirs, which are characterized by steep side slopes above relatively flat bottoms, most of the sediment is relatively coarse and heavy and is deposited near the head of the reservoir in well-defined steep-faced deltas close to the old stream channel rather than uniformly distributed over the reservoir bed and sides, as usually occurs when the incoming sediment is fine or largely colloidal. The characteristic wide-season variation in pool level and consequent movement of the head of pool up and down the canyon causes these delta formations to occur at widely varying points along the stream, but does not cause deposition on the side slopes. Therefore, sediment surveys can usually be limited to the flat bottom areas, at least for reconnaissance purposes. The scope of the survey can sometimes be further limited to a comparatively few ranges, because examination of the operational record of the reservoir will often indicate that most of the sediment accumulation should be found within a particular range of elevations.

The typical wide fluctuation of reservoir level that occurs in these reservoirs, because most of the annual flow is concentrated during one season of the year, also makes it possible to carry out a large part of a reconnaissance survey above water, if the survey is properly timed. Under these conditions it is sometimes possible to determine by aerial observation and aerial photographs or by aerial photographs, particularly

in color, where and when appreciable sediment has been deposited. Such aerial observations are very inexpensive and may be entirely adequate when only general information is required. If more detailed information is required, these observations may also indicate where additional surveys are needed.

For reconnaissance of the part of the reservoir area that remains under water, use can be made of a spud bar that will measure sediment depths of 10 to 15 feet. Figure 1 shows the spud bar used by the Sacramento District Corps of Engineers. This bar, which is 10.5 feet long and 1¼ inches in diameter, was manufactured at a local machine shop and is basically the same as the bar designed and used by U.S Soil Conservation Service. Spud bars are best adapted to water depths of less than 100 feet but have been successfully used in depths of over 200 feet. Special precautions must be taken when working in deep water to prevent loss of part or all of the sample while the bar is being pulled up. The survey should be made in calm weather when there are no large waves to cause excessive surging. The spud bar should be raised slowly so as to wash the sample as little as possible. Coating the bar with heavy grease prior to each measurement has been found to aid in retaining the sample; by this means samples have been obtained in water over 200 feet deep. The small grooves on the outside of the tip and the cup in the bottom of the tip are very helpful in retaining samples of the material at the deepest point penetrated.

Pine Flat Reservoir on Kings River, Calif., has been selected as an example to show the use of various reconnaissance techniques. This reservoir is located at the base of the foothills about 30 miles east of Fresno. The reservoir is created by a concrete gravity-type dam 440 feet high, with a crest length of 1,820 feet. Flows are released through two tiers of 5- by 9-foot conduits or over a gated spillway. The lower tier of outlets is located just above the riverbed at elevation 570 feet, and the upper tier is located 140 feet higher. The reservoir

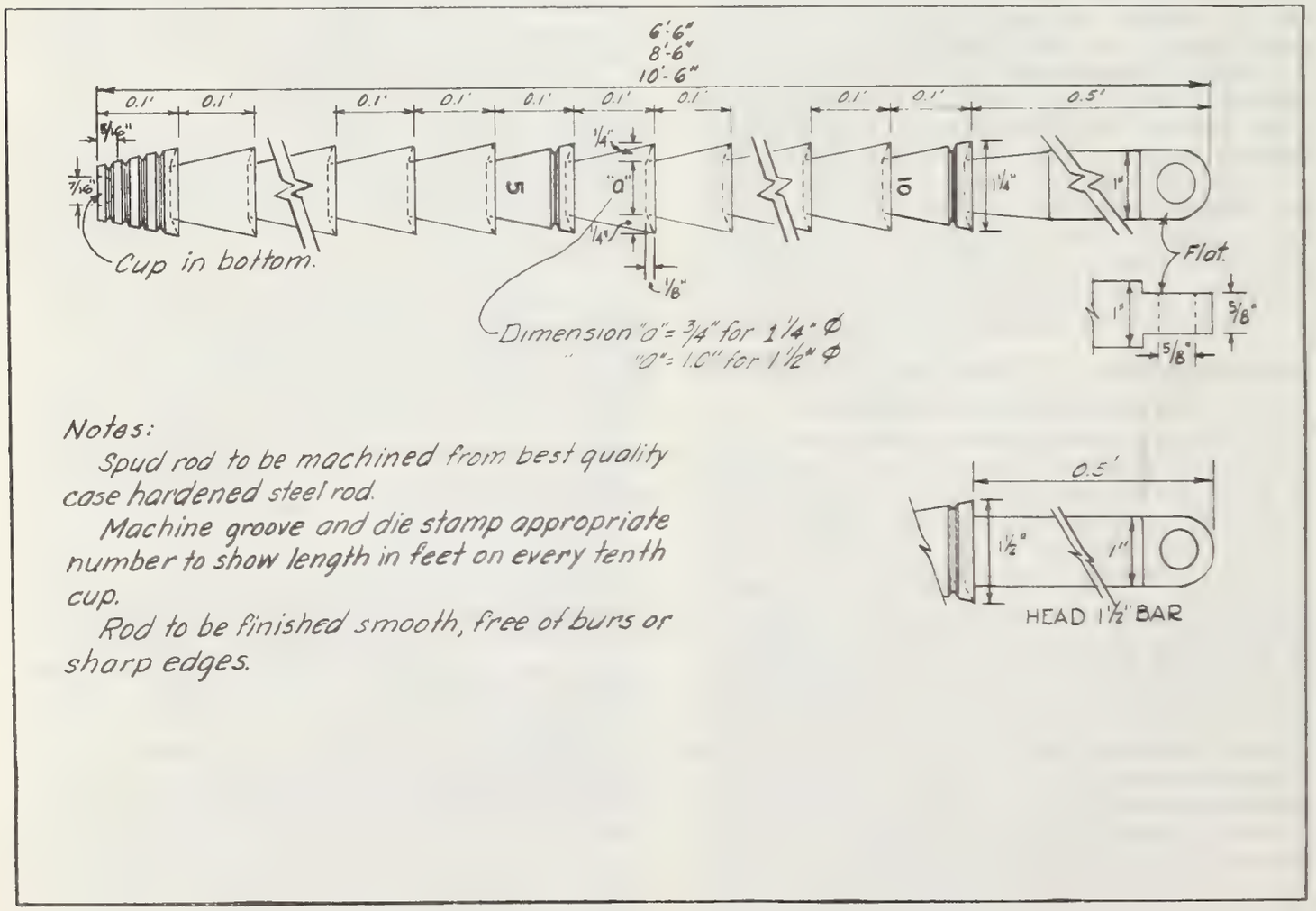


FIGURE 1. — Spud rod for sediment sampling.

was placed in full operation in 1954. Since that time, releases have been made only through the upper tier of outlets (or over the spillway since 1955), except for a short period in the summer of 1962. The reservoir at elevation of 951.5 feet m.s.l. contains 1 million acre-feet, covers an area of about 6,000 acres, and is 20 miles long. Reservoir operation is for flood control and conservation.

The drainage area above Pine Flat Reservoir covers 1,542 square miles and ranges in elevation from less than 1,000 feet to over 14,000 feet, with 65 percent of the drainage area above 6,000 feet. Mean annual runoff is about 1,650,000 acre-feet, of which about 1,190,000 acre-feet results from snowmelt. Inflow has ranged from 65 c.f.s. to 110,000 c.f.s. Annual rainfall ranges from about 15 inches in the foothills to over 40 inches at the higher elevations. Most of the precipitation falls as snow above about 6,000 feet. The higher mountain areas above 10,000 feet are characterized by little soil cover and large areas of exposed granite, whereas a large part of the area between 10,000 and 5,000 feet has deep soils and dense forest cover. The area below about 5,000 feet is generally covered with brush and grass.

A study of probable sediment rates in various proposed reservoirs in the Sacramento District was made for the Corps of Engineers by the U.S. Soil Conservation Service in 1947. Results of this study, which was reported in U.S. Soil Conservation Service's Special Report No. 10, included an annual sedimentation estimate for Pine Flat Reservoir. Annual sediment inflow to Pine Flat Reservoir was estimated at 230 tons per square mile of drainage area. The trap efficiency of the reservoir was estimated at 96 percent and the specific weight of deposited sediment as 62 pounds per cubic foot. The annual deposition in the reservoir was therefore estimated at 220 tons or 0.16 acre-foot per square mile or about 250 acre-feet per year. This would amount to only about 12,000 acre-feet of deposition in 50 years, or slightly over 1 percent of the reservoir capacity.

Although sediment deposition in the reservoir did not appear to be a major problem that might

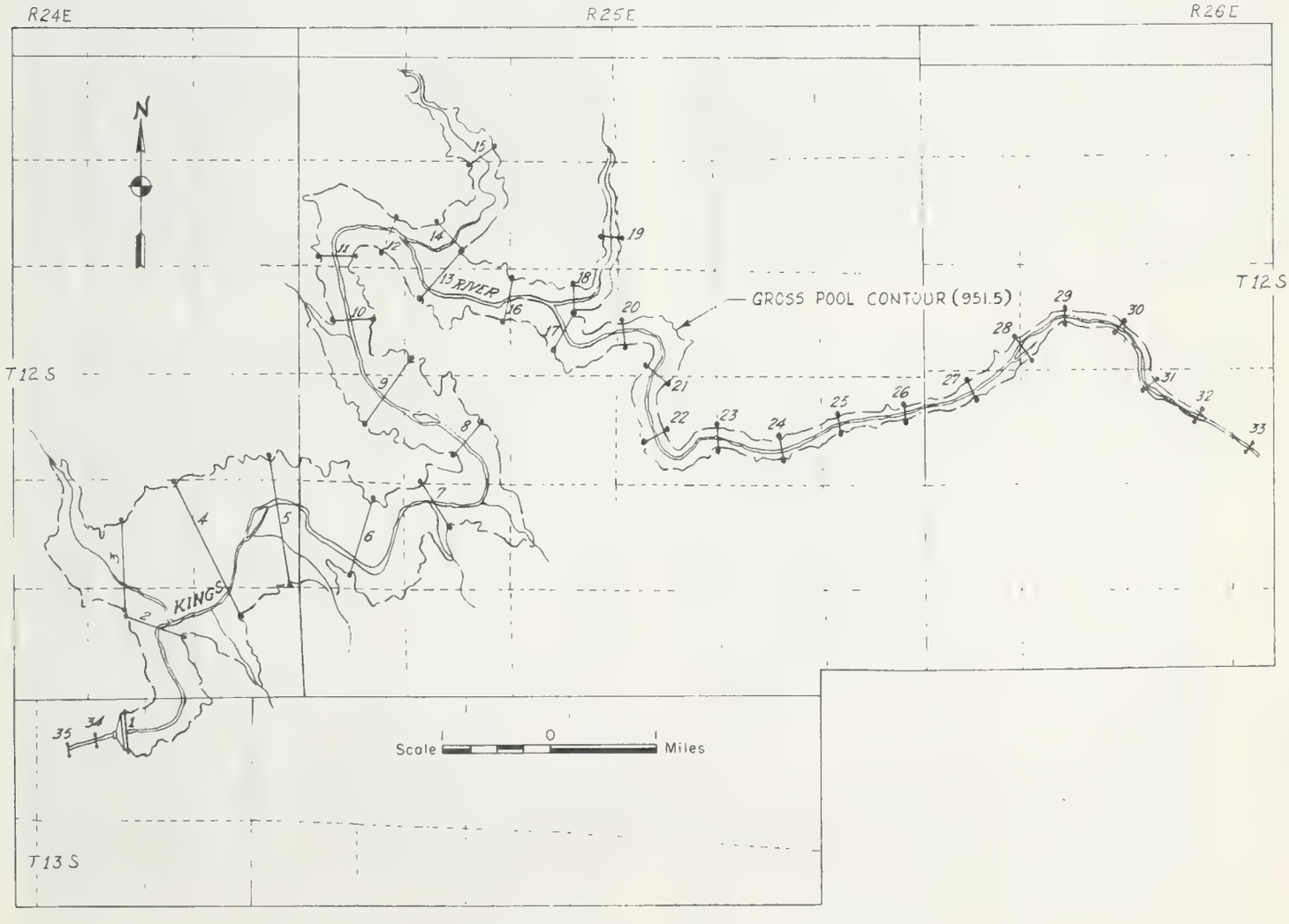


FIGURE 2. — Sediment ranges, Pine Flat Reservoir.

immediately affect reservoir operations, it was decided to establish a network of permanently monumented sediment ranges and to resurvey then at sufficiently frequent intervals to detect any gross errors that may have been made in estimates of quantity or location of deposits and as a means of measuring very long range sedimentation and changes in the reservoir capacity. This was highly desirable, since good sediment records showing both location and amount of sediment were not available for any comparable reservoir in the Sierra Nevada area. A system of 31 ranges was established in the reservoir, as shown on figure 2.

It also was decided to measure sediment outflow regularly, in order to verify the trap efficiency of the reservoir. This is accomplished by taking "grab" samples in the extremely turbulent area below the flip bucket. Comparison of several grab samples taken in this turbulent area indicate that each sample is representative and that integration of several samples is not required. Also, the same quality of results can be expected with grab samples under this condition as would be obtained with detailed samples of a normal river channel with the use of a standard sampler, such as a P–46. Samples are taken once each month, with special samples taken each time a sudden major change is made in the outflow, or if unusual turbidity occurs. Suspended sediment has varied from less than 1 part per million by weight to about 140 parts per million, with the latter value observed for only a short period during the 1955 flood when the reservoir was quite turbid. Normally, the reservoir outflow is clear, with a turbidity of only 2 to 8 and suspended sediment of 1 to 3 parts per million. The calculated annual sediment outflow has been less than 5 acre-feet per year, thus indicating that the trap efficiency is probably in the order of 96 percent, as had been estimated by the U.S. Soil Conservation Service in their Special Report No. 10. This sampling program will be continued and reevaluation of the trap efficiency will be made at the time of the next complete resurvey of the reservoir.

While the estimated annual rates were believed reasonable, little was known of the amount of sedimentation that might occur during a single major rain flood. It was therefore decided to make a special sedimentation survey following the major rain flood of December 1955. Although a second rain flood occurred in January 1956 before the survey could be made, the reservoir stage during the January flood was about 150 feet higher than during the December flood. Since it was believed the sediment from each flood would be deposited mainly near the head of the current reservoir pool, it therefore appeared likely that the results of each flood would be largely separable, and if not, the total amount in the reservoir could still be obtained.

A reconnaissance survey was conducted in September 1956 while the reservoir was at its lowest stage since the occurrence of the December 1955 flood. Reservoir stages from beginning of storage in 1952 to October 1962 are shown on figure 3. Reservoir stage was at about elevation 740 at the time of the peak inflow in December 1955, and at elevation 875 at the time of the peak inflow of the January flood, and subsequently reached a maximum elevation of 941

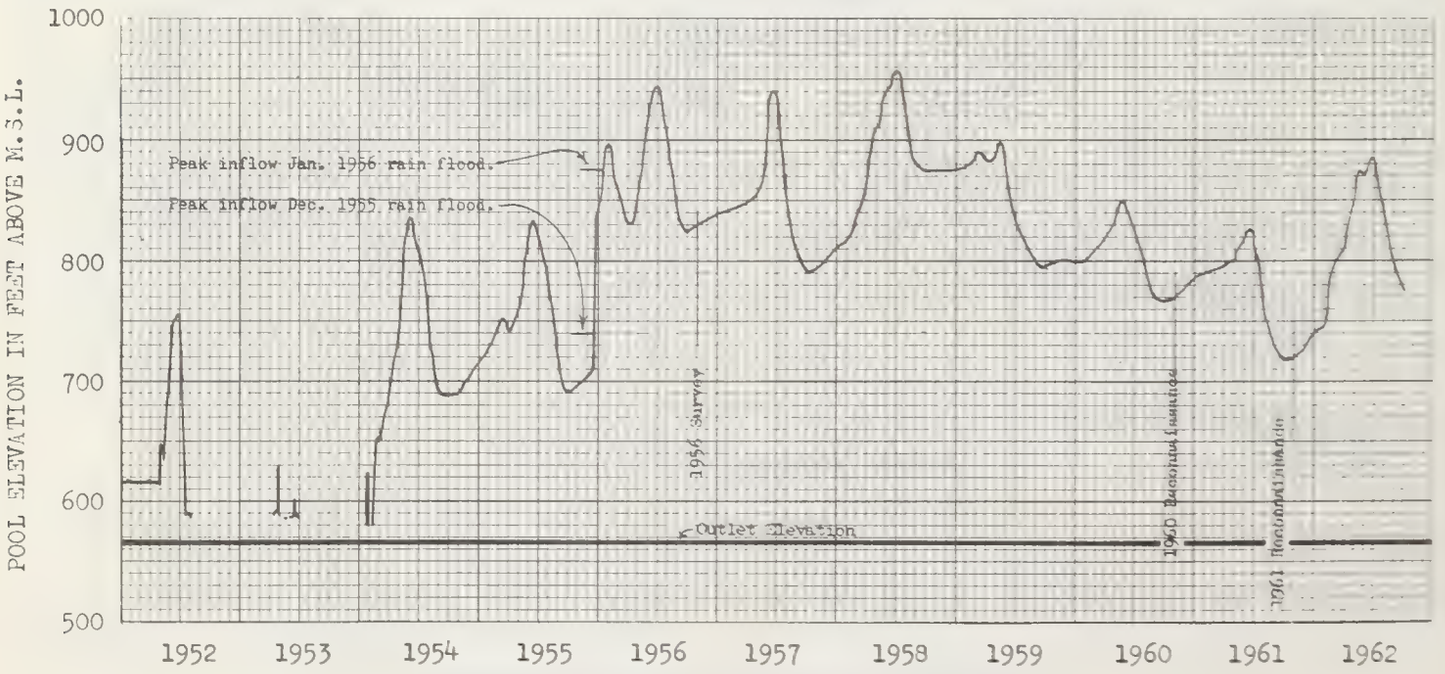


FIGURE 3. — Pine Flat Reservoir pool elevations.

feet during the spring of 1956. The reservoir had receded to elevation 826 in September 1956 when the reconnaissance survey was made.

All ranges above streambed elevation 826 were inspected and observations and estimates of deposition depths and amounts made. Depth of sediment was determined by digging with either a shovel or soil auger. Distances along the ranges were estimated from the reservoir topographic maps and the original detailed surveys of the sediment ranges. The portion of the reservoir area below elevation 826 was explored by use of the spud bar along the sediment ranges. Care was taken to sample at the deepest point along each range so as to be certain of obtaining samples in or near the old river channel. The bar was attached to 300 feet of copper core tiller line. The line was tagged at 10-foot intervals, and a large float was attached to the end so the line could not accidentally be lost when a sounding was made. The spud bar was lowered from a boat until bottom was found, then raised a few feet, and gently dropped to detect rocks or coarse gravel.

After locating a sampling spot free from such rocks and gravel, the bar was raised to the surface and dropped or plunged to obtain optimum penetration. The total depth from water surface to the point of the bar was measured by the tag line both before and after dropping for penetration. A simultaneous measurement of water depth at each point was made with a 3-pound bell sounding weight and standard tagged sounding line as an additional check on penetration depth. The differences in the two depths thus gave a check on depth of penetration of the spud bar. It was found that the bar would not penetrate more than about 1 foot of natural ground, but was capable of penetrating up to 10 or more feet in silt deposits. In nearly every instance where the spud bar penetrated to natural ground, samples of the material penetrated were retained even after withdrawal through over 100 feet of water. Coating the bar with grease aided in retaining the samples in deeper water. Therefore, even where deep water made it difficult to retain a sediment sample, it was usually possible to determine the approximate depth of sediment, either by examination of the retrieved bar sample, or by depth comparison.

Upon completion of the reconnaissance surveys, sediment depth at each sampling point and the position of each point on the cross section were plotted for all ranges surveyed and an estimate was made of the total sediment in the reservoir. This estimate indicated total stored sediment in the reservoir was probably between 1,300 and 2,100 acre-feet; thus, indicating a sedimentation rate of about 400 acre-feet per year. The average annual sediment rate had been estimated at 250 acre-feet per year, but since this 4-year period of measurement included the very large flood of 1955, it seemed reasonable that the amount deposited would exceed the estimated average annual rate.

The reconnaissance survey indicated that sediment deposits along many of the sediment ranges were great enough to be effectively measured by soundings along the ranges. It was decided to make a detailed survey of every range in the reservoir except ranges 3 through 7, which were omitted since the reconnaissance indicated no measurable sediment in these ranges. The detailed survey utilized chained cross sections above the pool level and lead line soundings, by means of boat and tag line within the pool. Lines of equal sediment depths were plotted as shown on figure 4 and the volume of sediment was computed by planimetering the areas so defined. The total sediment volume was thus calculated to be 1,450 acre-feet, which was within the range of the reconnaissance estimate. The deepest deposits were found near range 16, which crosses the reservoir bottom at elevation 730, just below where the tip of the reservoir was at the peak inflow during the December 1955 flood. Figure 4 shows the heavy deposits between range 13 and ranges 17 and 18. Of the total 1,450 acre-feet of sediment, about 420 acre-feet, or 30 percent, was found between ranges 12 and 17 and apparently was mostly deposited during this large flood.

From 1956 to 1960 reservoir stages remained relatively high and no resurveys of any kind were attempted. However, in December 1960, reservoir drawdown reduced the stage to about 773 feet, 50 feet below the level at the time of the 1956 survey. A ground reconnaissance was made and it was observed that there had apparently been considerable downstream movement of sediment from the upper ranges and most of the sediment above the tip of the reservoir (at range 17) had moved downstream and formed a very long delta along the river bottom. This delta varied in width from about 400 feet on the downstream end to less than 100 feet at the upstream end and appeared to be somewhat unstable and continually progressing downstream as the reservoir receded. Flows were distributed evenly over this delta and had not formed narrow cuts in it as might be expected. This was believed due to the fact that reservoir inflow had been very low during the drawdown period, and it was felt that such channels would be cut and the silt and sand carried further down the reservoir when flows increased. Observations during the winter of 1960 showed this to be true, and considerable sediment movement was noted as normal winter and spring flows came into the reservoir.

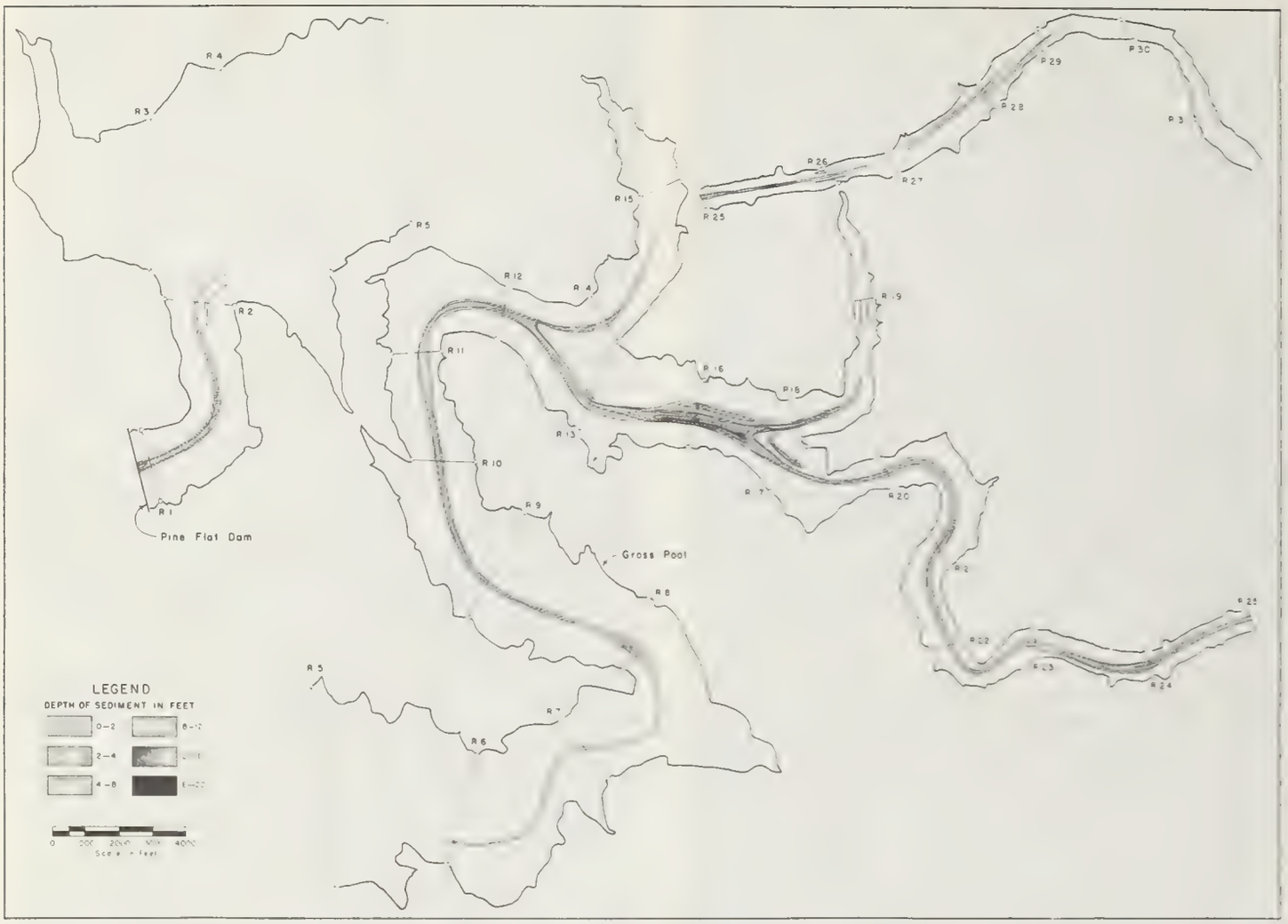


FIGURE 4. — Sediment deposition in Pine Flat Reservoir, May 1952 to November 1956.

During the summer of 1961 the reservoir was drawn down to elevation 718, the lowest stage since the summer of 1953 and 20 feet below the pool level that existed at the peak of the 1955 flood. This provided an excellent opportunity to resurvey the reservoir area above the pool level by use of reconnaissance methods. Aerial observations and photographs indicated that, as in 1956, most of the sedimentation was on the relatively flat reservoir bottom, with only very minor amounts on the steeper hillsides. Figures 5 and 6 are aerial photographs of the reservoir areas in the vicinity of ranges 16, 17, and 22, with the range lines superimposed on the photographs. Stumps, rocks, and gravel can be seen on the hillsides and on the outside of the river bends, thus indicating no deposition. Sandbar formations can be seen in the old channel and sediment of up to 10 feet deep was subsequently measured on range 17. The aerial observations also indicated that in certain reaches of the reservoir more deposition had occurred between ranges than along the ranges. In planning the range layout, more consideration was given to locating the ranges where they would be representative of sediment deposits after a long period of reservoir operation, such as 50 years, than during the first few years of operation. It was felt that ranges so located would also be reasonably representative of depths between ranges during the accumulating period and that sediment volume could initially be computed by methods other than the end-section method if that method appeared unsuitable.

For economic reasons consideration was given to placing range monuments at points where horizontal control points had already been established during the topographic survey of the reservoir. These points were often on "noses" and as a result the range lines between these points generally cross the reservoir at sections having somewhat less than the average width. Consequently, there are more off-channel flat areas in the reservoir bottom between the ranges than are represented by the ranges. Sediment

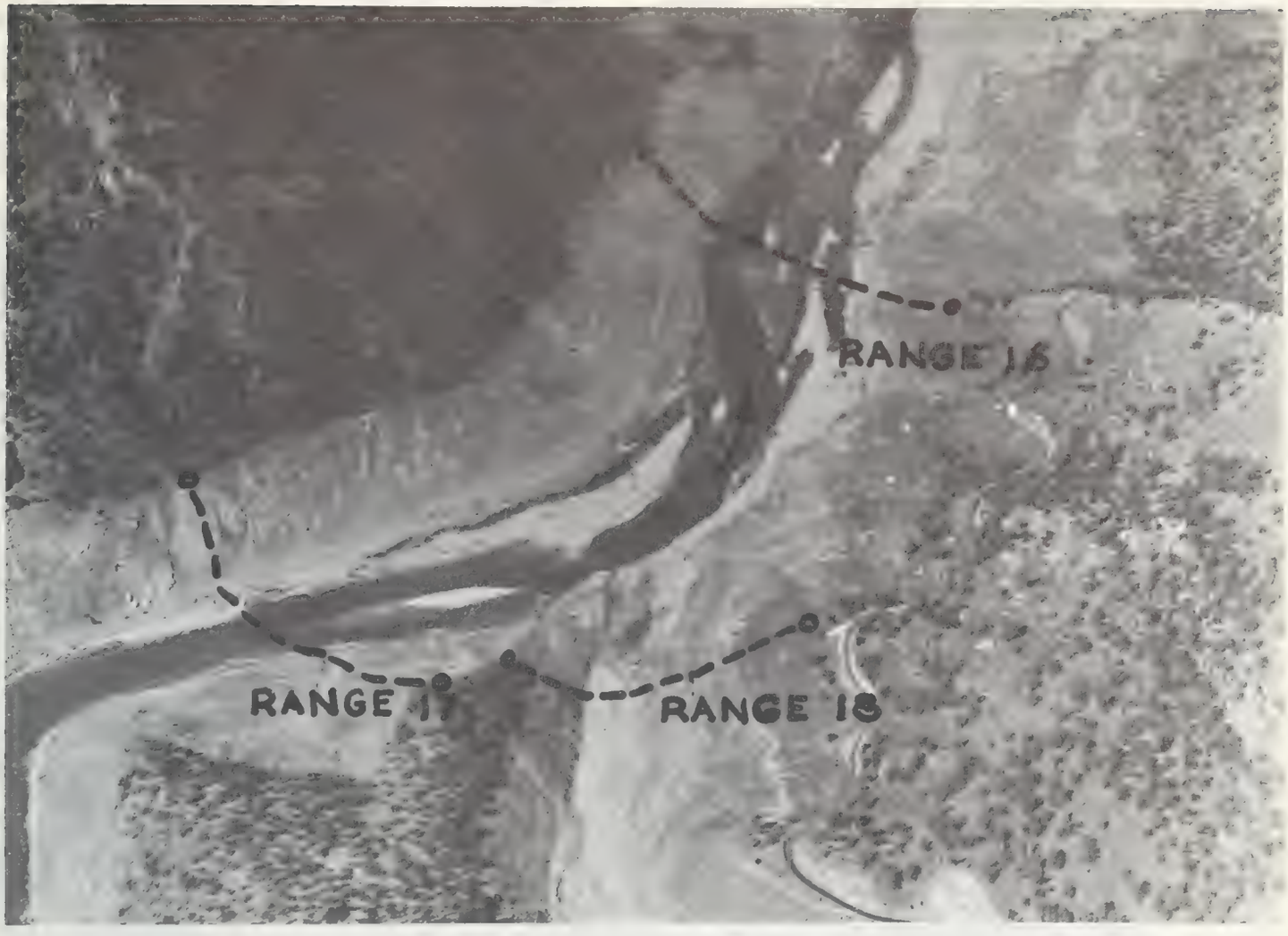


FIGURE 5. — Sediment deposits, ranges 16, 17, 18, Pine Flat Reservoir.

deposited in these off-channel flats or terraces is likely to be "benched" and not moved and redeposited during times when high flows occur and the reservoir is at stages below their level. Such was the case at Pine Flat and as a result, some sections of the reservoir have more deposition than had been represented by surveys along the existing ranges.

Preliminary ground reconnaissance of the entire reservoir above the water level confirmed the aerial observations, and a reconnaissance type resurvey of all ranges above the reservoir was made. As in 1956, depths were obtained by digging through the sediment with a soil auger or shovel and by observing depths of vertical cut banks where possible. Position of the various points on each cross section was determined by estimating distance from recognizable points on the cross sections and the reservoir topographic map. Many points between the ranges were also examined and sediment depths plotted on the reservoir topographic map. Lines of equal sediment depth were then plotted on the reservoir map as shown on figure 7 and planimetered as after the 1956 survey.

It was assumed that only minor deposition had occurred below range 11 between 1956 and 1961, since the reservoir had been considerably above that level during the entire period prior to the 1961 drawdown. The total sediment deposit so measured was about 1,950 acre-feet, indicating additional deposition of about 500 acre-feet since the complete survey of 1956. This deposit of only 500 acre-feet in 5 years or about 100 acre-feet per year is less than half the predicted annual amount. However, the total of 1,950 acre-feet in the 9 years of reservoir operation shows an average annual sediment deposition of about 220 acre-feet, very near the original prediction of 250 acre-feet.

Figures 8 and 9 are reservoir profiles which show maximum observed sediment depths on the various cross sections during the 1956 and 1961 surveys, respectively. Comparison of these profiles shows that most of the sediment accumulated since 1956 is above range 17, but below

range 23. It can also be noted that at least a part of this sediment was probably eroded from the area between ranges 23 and 27, where it was deposited prior to 1956. Examination of figures 4 and 7 also show that erosion of sediment occurred above range 23 and additional deposition occurred below range 23.

Although the main advantage of a reconnaissance survey is lower cost, poorer results might be expected, but this may not always be true.

Inspection of figures 4 and 7, particularly in the vicinity of ranges 13, 14, and 15, indicates that a more accurate picture may sometimes be obtained by a reconnaissance survey than from

FIGURE 6. — Sediment deposits, range 22, Pine Flat Reservoir.

a range survey, due to the examination of areas between the ranges. It is realized that where sediment deposits are of great depth and are uniformly distributed this would not be the case. Only 6 man-days were required in the field for the 1961 reconnaissance survey and about the same amount of time was spent in office analysis. Thus, the reconnaissance survey was accomplished for about $500 as opposed to the estimated cost of $10,000 for a detailed resurvey of all ranges, using standard survey procedures. In the case of Pine Flat Reservoir, it appears highly improbable that the 1961 estimate could be as much as 50 percent in error

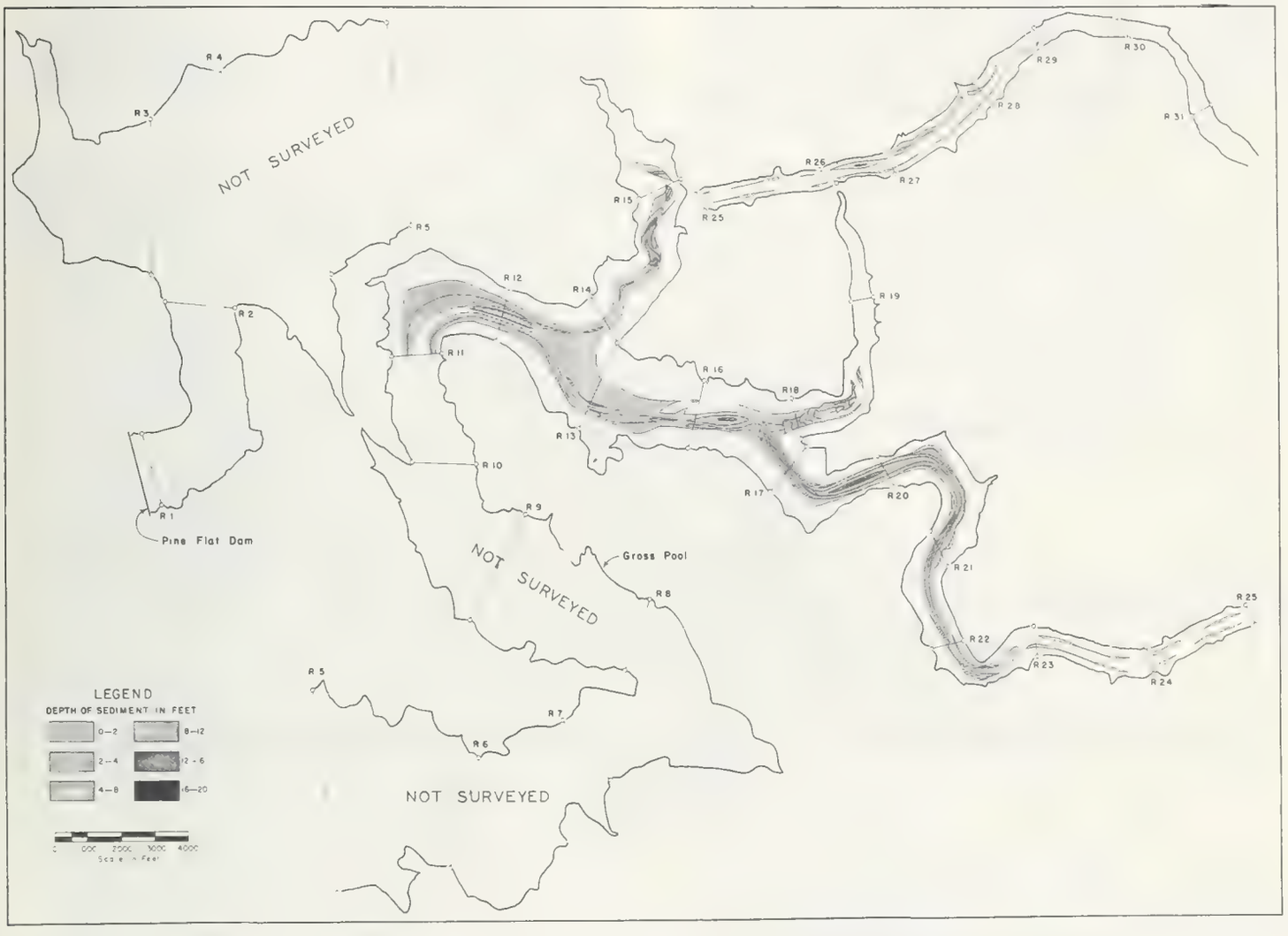


FIGURE 7. — Sediment deposition in Pine Flat Reservoir, May 1952 to November 1961.

and the results are considered adequate for project requirements.

From these studies of sedimentation in Pine Flat Reservoir, it is concluded that sufficiently accurate results may often be obtained by reconnaissance surveys in reservoirs where (1) rates are relatively low, (2) total capacity is large, and (3) current information is required to determine whether inlet or other project works are likely to be affected by sedimentation. Many reservoirs in the Sierra Nevada Ranges in California probably are in this category. It is believed other reservoirs in other areas of the country will have these same general characteristics. In such cases it appears that reconnaissance surveys can be used as an economical method of making interim surveys to supplement more detailed surveys made at infrequent intervals. It is further concluded from these reconnaissance surveys that, in reservoirs of this type, care must be taken in planning a range layout or data obtained from the ranges alone may not be representative. Also, reconnaissance surveys should be used as a method of checking the adequacy of a range system early in the life of the project.

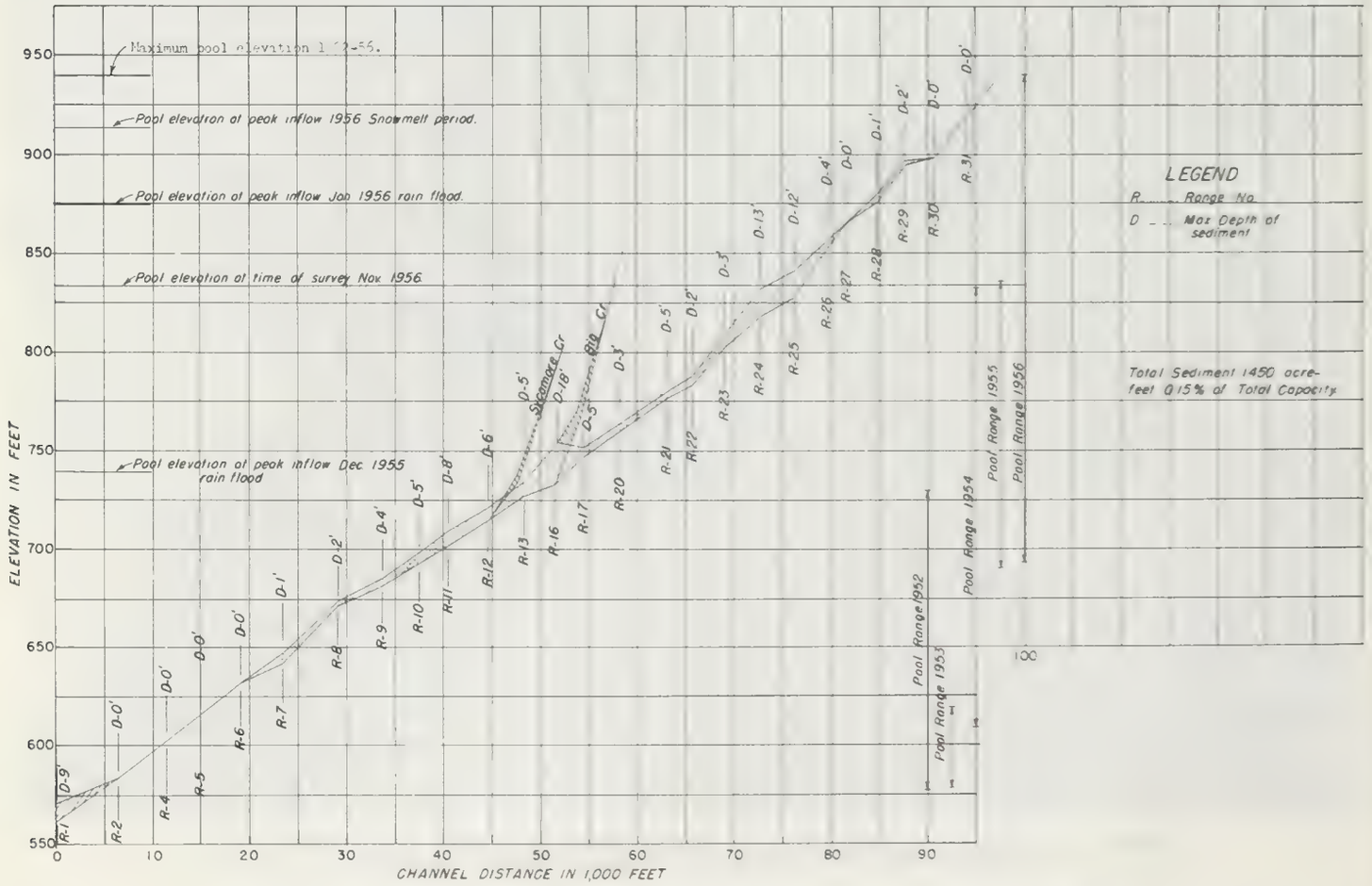

Maximum pool elevation 1 2-56.
Pool elevation of peak inflow 1956 Snowmelt period.
Pool elevation of peak inflow Jan 1956 rain flood.
Pool elevation at time of survey Nov. 1956
Pool elevation of peak inflow Dec. 1955 rain flood
LEGEND
R. . . Range No.
D . . . Max Depth of sediment
Total Sediment 1450 acre-feet 0.15% of Total Capacity
Pool Range 1952
Pool Range 1953
Pool Range 1954
Pool Range 1955
Pool Range 1956
Sycamore Cr.
Big Cr.
ELEVATION IN FEET
CHANNEL DISTANCE IN 1,000 FEET

FIGURE 8. — Distribution of sediment in Pine Flat Reservoir, November 1956.

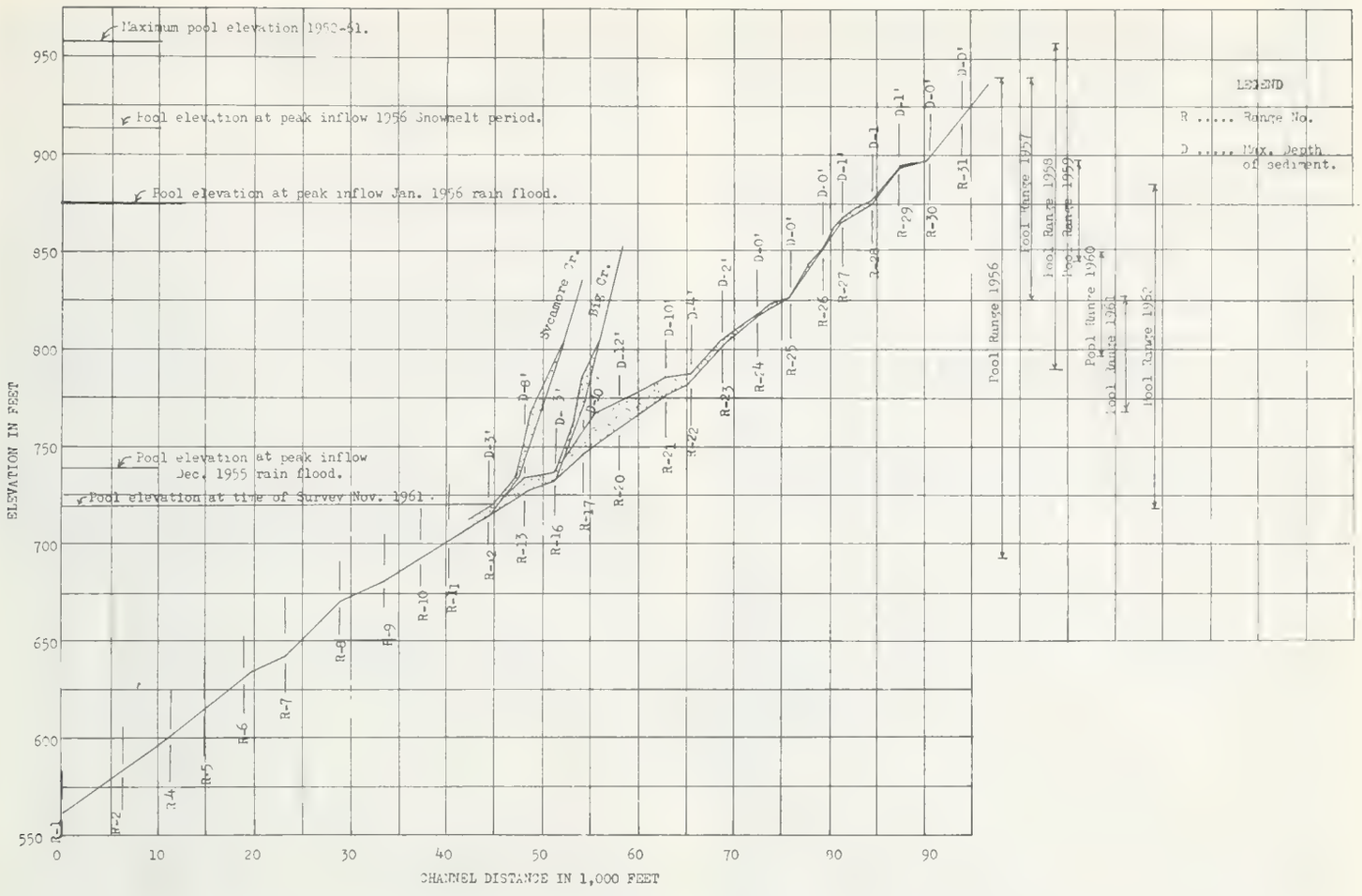

Maximum pool elevation 1952-61.
Pool elevation at peak inflow 1956 Snowmelt period.
Pool elevation at peak inflow Jan. 1956 rain flood.
Pool elevation at peak inflow Dec. 1955 rain flood.
Pool elevation at time of Survey Nov. 1961.
LEGEND
R ..... Range No.
D ..... Max. Depth of sediment.
Sycamore Cr.
Big Cr.
Pool Range 1956
Pool Range 1957
Pool Range 1958
Pool Range 1959
Pool Range 1960
Pool Range 1961
Pool Range 1962
ELEVATION IN FEET
CHANNEL DISTANCE IN 1,000 FEET

FIGURE 9. — Distribution of sediment in Pine Flat Reservoir, November 1961.

RECONNAISSANCE-TYPE RESERVOIR SEDIMENTATION SURVEYS

[Paper No. 85]

By ALBERT L. HILL, *hydraulic engineer, U.S. Army Engineer District, Kansas City*

Synopsis

Reconnaissance-type reservoir sediment surveys can be made at a reasonable cost and with sufficient accuracy to meet many operational and administrative needs. This technique should become more effective as additional experience is gained with reservoirs of various sizes and with the types of equipment now available. Three reconnaissance-type surveys were made in the Kanopolis Reservoir on the Smoky Hill River in Kansas. Point soundings were made on selected ranges and the results plotted on the original cross sections. The sections were completed by extrapolation and the sediment volume computed. Results of these three surveys were found to be from 1 to 10 percent higher than the estimates from sediment inflow-outflow data. A complete survey made in 1960 showed the sediment volume to be 14 percent higher than that estimated from the inflow records. This kind of agreement indicates an acceptable degree of accuracy for the reconnaissance-type survey.

Introduction

The larger reservoir projects and many small and medium projects have sediment-observation programs that include inflow and outflow sampling stations and a system of sediment-observation ranges, or a substantial part of these provisions. A complete resurvey of sediment ranges may cost from $0.25 to $0.50 per acre, depending on amount of exposed soft sediment, spacing of ranges, and the extent of weeds, woody growth, or field crops that may require clearing a part of some ranges. An abbreviated-type survey may consist of the "armchair" survey, computing inflow and outflow and estimating the location of deposits from reservoir-stage

records, or the reconnaissance-type surveys consisting of (1) checking exposed sediment depths and submerged deposits' thickness by probing, (2) visual inspection of exposed deposits and cruising the permanent pool with an echo-sounder that will reflect sediment thickness, (3) complete survey of a few selected ranges, and (4) a number of point observations of sediment depths in the permanent pool and exposed deposits.

The principal reasons for making the abbreviated- or reconnaissance-type survey are (1) low cost, (2) evaluating the need for a complete survey, (3) for administrative purposes at a multiple-purpose project where different storage allocations are involved, (4) measuring sediment deposits at boat harbors and launching facilities, and (5) investigating mud flats or swamp areas.

Three of the reconnaissance-type surveys were made in the Kanopolis Reservoir, situated on the Smoky Hill River in central Kansas, as interim observations. The Kanopolis Reservoir was constructed by the U.S. Army Engineer District, Kansas City, and has been in operation since 1946, primarily for flood control. The total initial storage capacity was 450,000 acre-feet, of which 53,000 acre-feet was for conservation and sediment requirements. At full-pool elevation of 1,508 feet m.s.l., the reservoir has an area of 13,900 acres and is 25 miles long. The conservation-sediment pool, at elevation 1,459 feet m.s.l., has an area of 3,550 acres and is 12 miles long. Prior to filling the reservoir, a series of 27 sediment ranges were established and surveyed across the main reservoir and 10 ranges were established and surveyed across tributary arms.

Suspended Sediment

Prior to construction of the dam, a sediment-sampling station was established at the stream-gaging station at Ellsworth, Kans., located immediately upstream from the reservoir on the Smoky Hill River. Sampling the outflow from the dam was started soon after impoundment began. These data were used to estimate inflow and outflow tonnage and, together with particle-size data, to compute volume of sediment deposits with due regard to age and degree of submergence.

Surveys

Fieldwork consisted of taking soundings at a number of points on selected ranges in the permanent pool and determining the thickness of deposits at selected points on that part of the ranges above the permanent pool. The monuments were located and flagged, and transit and stadia used to locate observation points on each range examined.

Office Work

Office work consisted of plotting field observations on the range profile and extrapolating to complete the section. The areas were planimetered and the volume of sediment computed by the end-area method. Results of the survey were compared with estimates prepared from inflow and outflow data. For the purpose intended, no further reports were necessary.

Results of Surveys

In 1960 a complete survey was made of the reservoir [1] and findings of the survey compared with estimates made from inflow-outflow data. Table 1 shows the results of the reconnaissance-type and complete survey as compared with estimates prepared from inflow-outflow data. In addition to volume comparisons, table 2 shows location determinations from the surveys. Results of the reconnaissance-type surveys agree very well with inflow-outflow data. Although the degree of accuracy of either is not readily apparent, taken together, they add up to reasonably convincing evidence that the results are reliable.

TABLE 1.—*Comparison of estimates from inflow-outflow data with survey results, 1960*

Date	Survey type	Estimated volume of sediment in Kanopolis Reservoir		
		Inflow-outflow data	Survey data	Inflow-outflow
		Acre-feet	*Acre-feet*	*Percent*
Nov. 1950	Reconnaissance	4,140	4,300	+104.0
Nov. 1951	Reconnaissance	7,100	7,130	+100.4
Sept. 1954	Reconnaissance	7,230	7,990	+109.5
Sept. 1960	Complete	12,370	14,380	+114.0

TABLE 2.—*Approximate percentage distribution of Kanopolis Reservoir sediment (by volume)*

Location	November		September	
	1950	1951	1954	1960
	Percent	*Percent*	*Percent*	*Percent*
In channel in conservation pool.	43	33	35	25
In channel upstream of conservation pool.	2	4	10	8
In overbank in conservation pool.	40	36	31	50
In floor of flood-control pool	15	27	24	17

Discussion

Results of this nature may be obtained by expenditure of a few hundred dollars for reconnaissance as compared to several thousand

[1] Results are contained in report "Sedimentation in Kanopolis Reservoir, Smoky Hill River, Kansas," prepared by U.S. Army Engineer District, Kansas City, dated December 1961.

dollars in the case of a complete survey on reservoirs comparable in size to the Kanopolis project. Improved equipment and methods now available should give even better results where the range method is used. For example, the U.S. Army Engineer District, New Orleans, has developed a piano-wire distance-measuring device that is valuable in measuring distances along the submerged part of the range. Annual maintenance of monument markers, so that they are readily located, helps to save time. In multiple-purpose projects, with on-line range monuments set near the water level of the permanent pool, a piano-wire distance-measuring device, and echo sounding equipment, a high-type reconnaissance survey is now possible. A complete reservoir survey involves a sizable sum of money. With a limited budget and the rapidly increasing number of reservoirs, it is necessary to resurvey only when the volume of sediment reaches the point where it affects reservoir operation or necessitates reallocation to the various uses.

Conclusions

The Federal Inter-Agency Sedimentation Committee has performed a very valuable service in collecting sediment data and distributing it to the various using agencies. In this connection, the collection of data sheets for all known sedimentation surveys is of great value. As the volume of sedimentation data increases and the individual investigator becomes more experienced, a fairly accurate picture of sediment accumulation in a reservoir may be constructed, based on the rather limited observations of a reconnaissance-type survey. Ingenuity in planning limited observations and sound judgment in extrapolating the data are the primary requirements for using the reconnaissance-type reservoir survey.

USING RAYDIST FOR SEDIMENTATION SURVEYS ON LARGER RESERVOIRS

[Paper No. 86]

By Isaac Shepherdson, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

The Garrison District of the Corps of Engineers purchased a single dimension type ER Raydist system in 1956 for the purpose of resurveying the many long sedimentation ranges on Garrison Reservoir. Raydist accomplishes distance measurement by the phase comparison of separate radio waves from two continuous wave transmitters, one that must be stationary and the other that is mobile. This system requires the simultaneous operation of three transmitting-receiving units, which consist of a master station, a shore station, and a mobile transmitter. The usual arrangement is to place the shore station at one end of the range line and operate the master and mobile stations from the sounding launch to record distances from the known shore station. Since only the distance from a known point is established, the centerline guidance of the launch is maintained by transit through radio voice communication.

Advantages of this equipment are (1) convenient horizontal control for sounding long ranges; (2) reduction in visibility problems; (3) continuous position determinations; (4) ability to reposition very closely; (5) increased accuracy; and (6) simplification of office computations. Disadvantages and limitations during calibration and operation of the equipment are (1) inaccuracies when the master station is located within 600 feet of the shore station or is near steep bluffs; (2) necessity for a restart when any unit fails during a sounding run; and (3) the need for a trained electronic technician to install and maintain the equipment.

The initial expense of this Raydist system necessitates a practical justification for use in sedimentation survey work. This requirement can be satisfied for such large reservoirs as Garrison, where some range lengths exceed 4 miles. However, for the normal operation on smaller reservoirs other methods of horizontal control, such as stadia, transit intersection, distance measurement by piano wire, or towing a current meter, must be considered as the more appropriate technique.

Introduction

Raydist was mentioned in an article published in the proceedings to the 1947 Inter-Agency Sedimentation Conference held in Denver, Colo.[1] This article led to an investigation of Raydist by the Omaha District as a means of measuring distance for surveying the sedimentation ranges on Garrison Reservoir.

At maximum pool elevation, approximately 72 of the ranges on Garrison Reservoir will have underwater lengths between 1 and 7 miles. Because of the large number of long ranges, sev-

[1] Discussion by Charles W. Thomas to article titled "Reservoir Sedimentation Surveys — Objectives and Methods," by Albert S. Fry. 1947 Fed. Inter-Agency Sedimentation Conf. Proc., Denver, Colo. 1948.

eral methods of horizontal control for resurveying these ranges were investigated.

The stadia and distance measurement by piano wire methods were satisfactory for ranges approximately 4,000 feet in length.

Towing a current meter[2] had been used for surveying long ranges; however, this method had disadvantages because of (1) time and computations necessary to prorate the error accumulated for a particular survey; (2) errors due to the effect of wind and waves on current meter readings; and (3) possible reruns when current meter distance differ substantially from the actual distance.

Transit and plane table intersection was considered adaptable for the survey of long ranges; however, there were disadvantages because of (1) necessary surveys and cost for establishing points for using this method; (2) problems due to visibility because of fog, haze, and in some instances timber; (3) the necessary computations (in the case of using the transit) for determining the stationing for the fix lines on the sounding chart; (4) reduced accuracy (in the case of the plane table) in transferring the stationing from the plane table to the fix lines on the sounding chart.

Raydist appeared to have several advantages over other methods for the sounding of long ranges: (1) convenient horizontal control for sounding long ranges; (2) reduction in visibility problems; (3) continuous position determinations, i.e., personnel in the sounding launch would know their position at all times; (4) ability to reposition very closely, i.e., resounding of parts of a range could be easily accomplished and following sounding of a range it would be easy to return to sampling locations selected from the sounding chart; (5) simplification of office computations; and (6) elimination of the cost of establishing points for cutting in the boat by transit or plane table.

No disadvantages in the use of Raydist for horizontal control on sedimentation surveys were discovered upon investigating its use by the Norfolk District, and the U.S. Lake Survey Corps of Engineers.[3] [4] Although the Lake Survey group did dispose of its type E Raydist system, its reasons for disposal did not apply to the work the Omaha district planned for the equipment. The purchase of Raydist was justified solely by the sedimentation surveys planned for Garrison Reservoir; however, it was envisioned that the Raydist system might be used on other reservoirs and by other Districts. Over one-half the initial cost of Raydist was justified by the elimination of the cost of establishing points for the use of the transit and plane table methods.

Consequently, in 1956 a single dimension range part of a type ER Raydist system was purchased from Hastings-Raydist, Inc. The equipment purchased was such that the additional components could be added to make a complete type ER Raydist system. Thus far, the Omaha district has not needed a two-dimensional system, because detailed hydrographic surveys of extensive areas have not been required. The district also contracted for a week's work by one of the Raydist technicians to help in installing, setting up, operating, and testing the equipment.

Briefly, Raydist accomplishes distance measurement by the phase comparison of the wavelengths from two continuous wave (CW) transmitters, one of which is moving and the other is stationary. The equipment literally counts the radio waves through which it moves. A single dimension Raydist system requires the simultaneous operation of three transmitters and three receivers and is made up of three distinct groups of equipment: the master station, shore station, and mobile (CW) transmitter (fig. 1). The phase meter in the master station measures distance in lanes. A lane is equal to one-half the wavelength of the mobile (CW) transmitter.

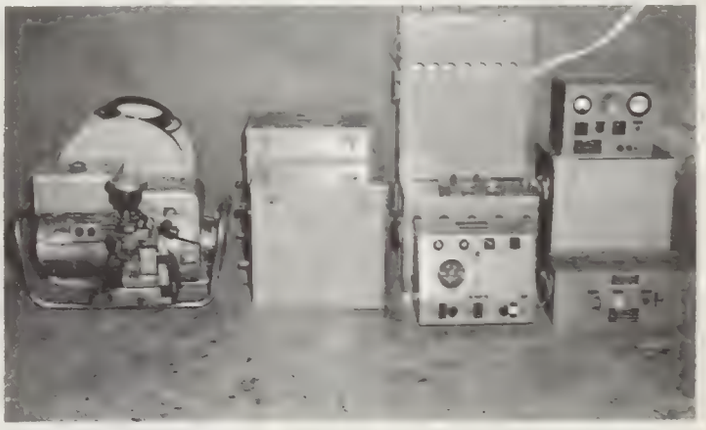


FIGURE 1. — Equipment comprising the three Raydist groups: Left to right are (1) generator and cable, (2) shore station units, (3) master station group, and (4) the test set, mobile transmitter and master power supply.

[2] Report of sedimentation survey, Denison Dam and Reservoir (Lake Texoma) by the Tulsa District, Corps of Engineers, June 1950.

[3] "Raydist for Shallow-water Hydrography," by Harold H. Waterfield, engineer, chief, Survey Branch, Norfolk District, and Cecil Hilliard, engineer, in the Norfolk District, November-December 1951, Military Engineer.

[4] "Raydist Preliminary Report," dated January 1, 1952, and "Final Report Test of the Type E Radist for Hydrographic Charting Over Fresh Water," dated June 30, 1953, U.S. Lake Survey, Corps of Engineers.

Arrangement of Raydist Units

The three basic groups of Raydist equipment are the master station, the shore station, and the mobile transmitter. These can be arranged to give three different position configurations: circular, elliptical, and hyperbolic.

The circular arrangement is achieved by operating with the mobile transmitter integral with the master station, i.e., it moves with the master station or remains fixed with the master station. In this arrangement either the master station or the shore station may be fixed or mobile and the phase meter at the master station will continually indicate the circular line of position upon which the mobile equipment is located. Lines of position are a family of concentric circles centered upon the fixed equipment. (At either group of equipment the measuring point, center of radiation, lies halfway between the CW transmitter antenna and the AM receiver antenna.) Figure 2 shows this arrangement and the resulting position configuration.

The elliptical arrangement is achieved by separating the mobile transmitter from the master station (forming three groups) and making the shore station mobile while the master station and mobile transmitter are fixed. The position configuration is a family of ellipses having their foci at the antennae of the mobile CW transmitter and the AM receiver in the master station. The phase meter will now indicate the ellipse line-of-position upon which the mobile unit is located. The ellipses are spaced a half-wave apart on the baseline extensions.

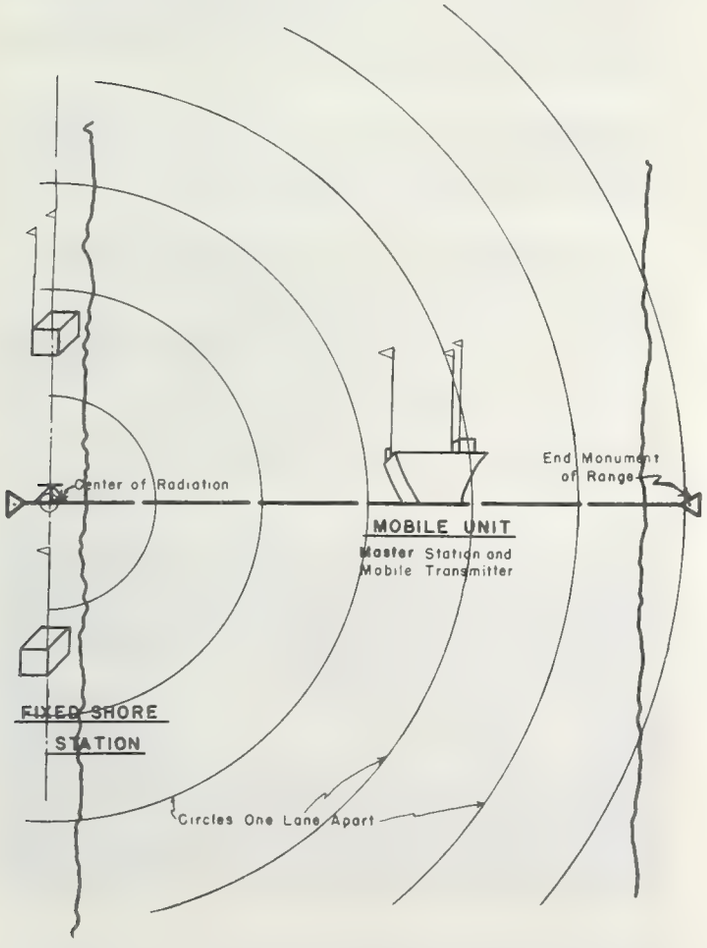


FIGURE 2. — Circular arrangement and resulting configuration pattern.

The hyperbolic arrangement is also achieved by using three groups of equipment. The mobile transmitter is used as the mobile unit, while the master station and the shore station are fixed. The indicator will now identify the position of the mobile unit on a family of hyperbolic lines of position having as their foci the AM receiver antennae at both fixed stations. The lines of position are spaced a half-wave apart along the baseline and gradually diverge with distance away from the baseline. In this case the baseline extensions also constitute lines of position (or lines of constant phase difference), and maximum and minimum phase meter readings will be obtained when the mobile transmitter travels (by any path) from any point on one extension to any point on the other extension. The difference of these readings will indicate the length of the baseline in lanes or $\frac{1}{2} f$. Figure 3 shows this arrangement and the resulting lines of position.

Pure range or true distance is measured along the line connecting the two or three groups of equipment comprising the circular and hyperbolic arrangements. The circular arrangement with the master station moving is the one most used, because position is shown at the master station for the mobile transmitter, which is a part of the master station in this case. The arrangement for the hyperbolic configuration has been used for control on long, shallow ranges, i.e., ranges that have areas too shallow for the draft of the large sounding launch.

Operating Principle of Raydist

The type ER Raydist system, as do all Raydist systems, employs the method of phase comparison of continuous wave (CW) signals. In the Raydist method, however, the phase comparison takes place at a radio frequency, thereby eliminating the problems of phase comparison at radio frequencies while maintaining the sensitivity and accuracy obtainable at the higher frequencies. This is accomplished by the use of two CW unmodulated transmitters operating on essentially the same frequency but slightly displaced by an audio amount. For example, one transmitter is operated at a frequency f and the second transmitter is operated at a frequency $f + \Delta f$, where Δf is an audio frequency

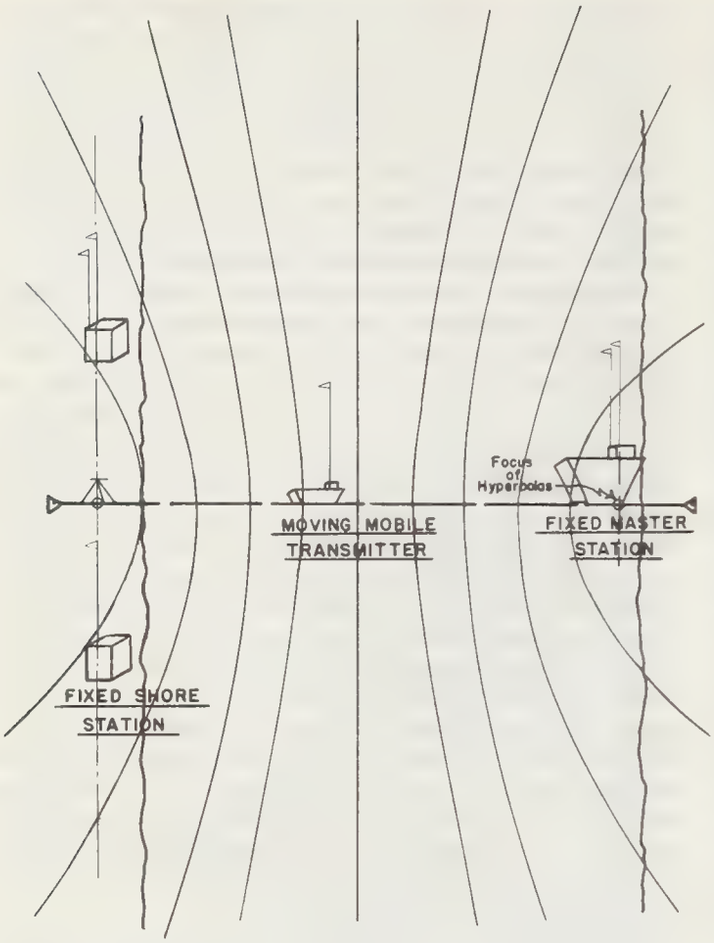


FIGURE 3. — Hyperbolic arrangement and resulting configuration pattern.

of usually 400 c.p.s. Receivers at the master station and shore station receive both carriers and detect a heterodyne or beat frequency, Δf. This heterodyne tone, as it is received at the shore station is returned to the master station and compared in the phase-measuring circuits, to indicate position. Since the heterodyne is derived from a fixed transmitter and a moving transmitter, its phase will be dependent upon the location of the mobile transmitter. Since both the master station and the shore station contain an AM receiver that must receive both of the CW transmitters and since both also contain those transmitters, it is obvious that each transmitter must be made to appear weak to the adjacent receiver but at the same time strong to distant receivers.

Special measures are required to permit these receivers to receive the strong local carrier and the relatively weak distant carrier simultaneously. This is accomplished by the use of nearly harmonically related frequencies and by making use of the second harmonic radiation at the shore station. For example, assume the primary basic frequency of the system to be f. This, then, is the frequency that determines the lane width of the system, i.e., the spacing of the lines of position. The mobile (CW) transmitter at the master station is operated on the frequency f and the (CW) transmitter at the shore station is operated on $\frac{f \pm \Delta f}{2}$. The AM receiver at the master station is a dual channel receiver that receives the local carrier f on one channel, and the distant carrier $\frac{f \pm \Delta f}{2}$ on the second channel. These two signals are combined in the receiver to produce a single heterodyne audio output of Δf. The AM receiver at the shore station is tuned to f and receives the distant carrier from the mobile (CW) transmitter as well as a small amount of the local transmitter's second harmonic radiation on $\frac{f \pm \Delta f}{2}$. The two signals heterodyne in the receiver to produce the audio beat of Δf. The audio tone, Δf, as detected at the shore station is then returned to the master station, via FM radio, for phase comparison. The results of the phase comparison are shown in lanes on the dial of the remote phase indicator. Figure 4 is a diagram of a two dimensional type ER Raydist system.

Radio frequencies allocated for the operation of this Raydist system are:

	Kc
Mobile CW transmitter	4,796.4
Shore station CW transmitter	2,398.0
FM transmitter	36.8

Transportation of Raydist Units

Shore Station

The shore station Raydist units and the necessary equipment to support the work required in setting up and operating the station are generally transported in a shallow draft outboard cruiser capable of independent operation from the larger sounding launch. A photograph of such a cruiser is shown in figure 5. This cruiser is also equipped with hull-mounted sounding transducers to permit sonic sounding of shallow areas and a sampling boom to permit sampling, velocity, or nuclear density probe measurements.

The shore station can be transported by vehicle; however, most of the shoreline along the Missouri River reservoirs is rough and irregular, with numerous bays that in many cases make it practically impossible to reach the range line. Transportation in a vehicle eliminates the versatility of being able to select the most desirable side of the reservoir for the shore station. On the other hand, an advantage of vehicle transportation is elimination of the error due to the master station coming close

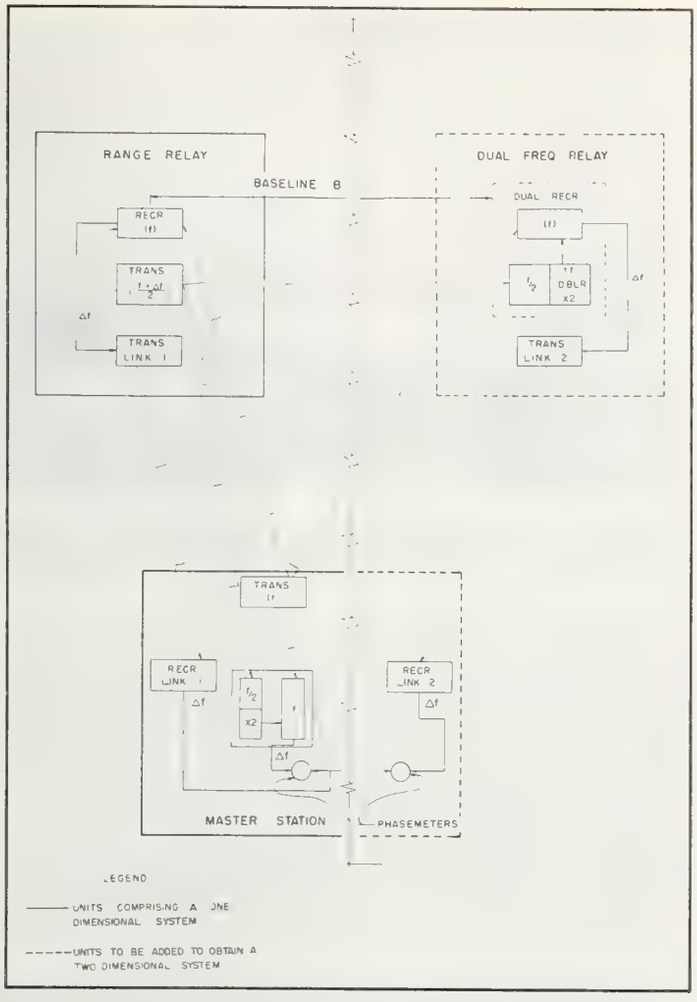


FIGURE 4. — Two dimensional diagram of the type ER Raydist system.

to the shore station (described under test procedure), because the shore station can be set up farther from the shoreline.

Mobile CW Transmitter

For the sounding of long deep ranges, and this pertains to approximately 95 percent of our sounding work, this transmitter is installed with the master station on the sounding launch. The CW transmitter cabinet is fastened in a high frame built for this purpose on the port stern corner. The transmitter's antenna is mounted on the side of the cabinet. Power in this case is obtained from the same 115-volt a.c. generator that supplies power to the master station.

FIGURE 5. — Shallow draft cruiser used for independent sounding operations and transportation of the Raydist shore stations.

For the sounding of long ranges that have areas that are too shallow for the sounding launch, this transmitter is installed in the outboard cruiser or smaller sounding skiff as necessary. Both boats are equipped with inboard tanks to accommodate the transducers of either an Edo or Bludworth sonic sounder. A 750-watt motor generator supplies power to the Raydist unit while a 12-volt storage battery supplies power to the sounder.

The Master Station

Since the master station serves as the focal point of operations for our Raydist surveys, it and the accessory components are installed on the sounding launch as previously described. The equipment layout varies for different launches, but in general the antenna schematic remains fairly constant as shown in figure 6.

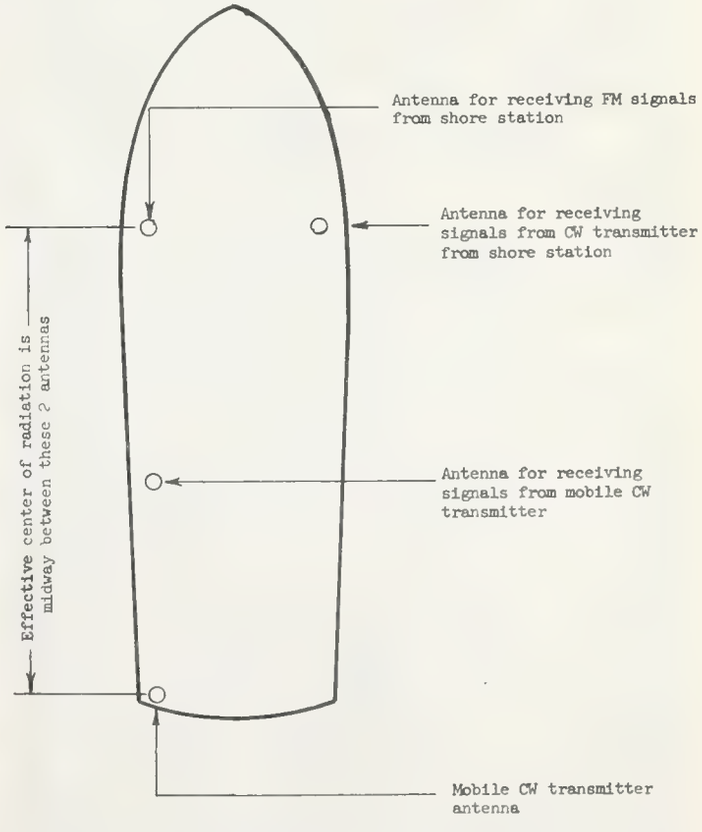

Antenna for receiving FM signals from shore station
Antenna for receiving signals from CW transmitter from shore station
Effective center of radiation is midway between these 2 antennas
Antenna for receiving signals from mobile CW transmitter
Mobile CW transmitter antenna

FIGURE 6. — General location for antennas for the master station on sounding launches.

Due to the use of the launch as a base of operations for all deep water observations, some of the components needed for Raydist operations serve dual functions. Perhaps the best illustration of this point is shown on the launch "Ellis" (fig. 7), which was designed specifically for reservoir sedimentation survey work.

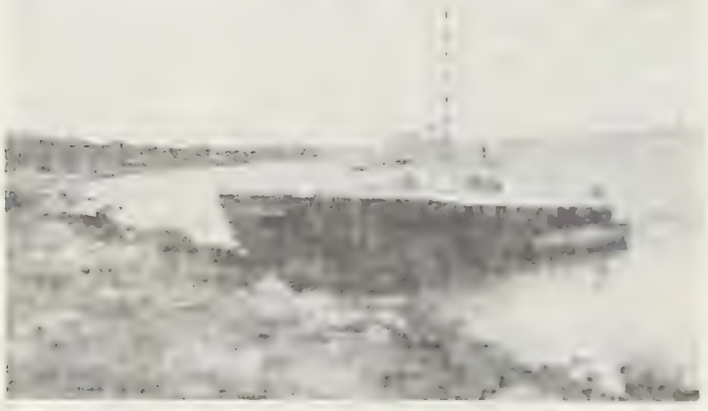


FIGURE 7. — Launch *Ellis* on Garrison Reservoir. Only bow booms were installed for this operation.

The "Ellis" is a 38-foot, steel-hulled work launch, propelled by twin diesel engines and containing a small galley, toilet, bunks for three personnel and a fairly large workroom. Other installed equipment includes dual controls, a two-frequency two-way FM radio, AM two-way radio, dial-type depth sounder, and a 3-kw. 115-volt a.c. diesel generator. Both 115-volt a.c. and 12-volt d.c. lights and outlets are installed throughout the boat. A heavy duty convertor is installed on each side to supplement and charge the batteries for each engine from the 3-kw. diesel generator. A flush-mounted power windlass is installed on the bow deck. For convenience in anchoring, 300 feet of ½-inch nylon line is carried on a shop-made reel. Two booms, equipped with B-50 stream gaging reels, are installed on the bow deck. These booms are used for sampling, for velocity measurements, and for bar checking the depth sounder. The upright members of the booms are used to mount two of the Raydist antennae. Two high booms, one 13 feet and the other 11 feet, are installed on each side near the stern of the boat. (See fig. 8 for typical installation on the launch "Dakota" used at Fort Randall Reservoir.) These booms, equipped with stream gaging reels, are used for taking density samples and for suspending the nuclear sediment density probe. All the booms can be rotated and have a locking device on the base to hold them in several positions.

FIGURE 8. — Launch *Dakota* on Fort Randall Reservoir. Note stern sampling booms and antenna arrangement.

A 15-foot triangular stadia board is mounted on top of the cabin. The Raydist master station is installed in a combination rack and table in the front port corner of the main cabin. The Raydist test set and depth sounder are mounted on top or on the table portion of this rack. The remote phase indicator is mounted in a revolving frame on the dash so that it can be turned or oriented for better operational viewing, particularly for the launch operator (fig 9).

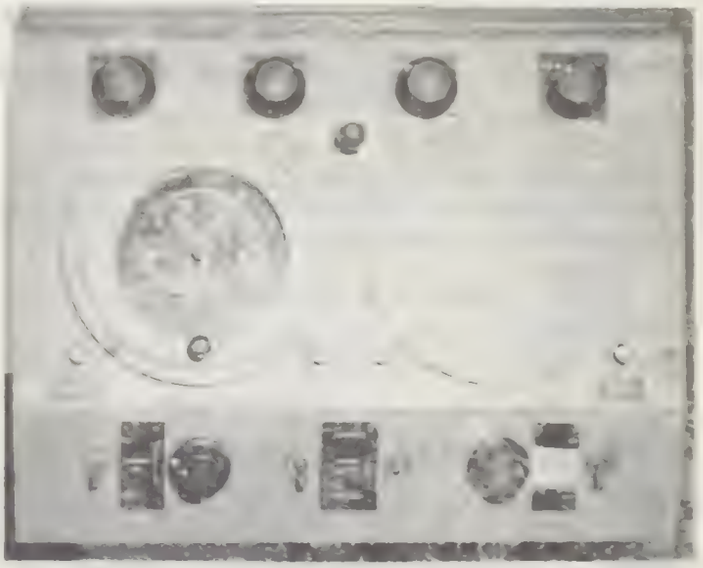


FIGURE 9. — Raydist remote phase indicator. For two-dimensional operation another dial is mounted in place of right cover plate.

Tests to Determine Raydist Performance

Hastings-Raydist, Inc. tested and operated the Raydist system on Chesapeake Bay near its factory in the presence of an electronic technician from the Garrison District. During this

testing, a distance of approximately 20 miles was measured, its repeatability was demonstrated, and distances to several known objects shown on maps of the Chesapeake Bay area were checked.

Prior to receiving the Raydist equipment, three sediment ranges near Garrison Dam and the town of Riverdale, N. Dak., were selected for checking and testing this equipment and lengths were accurately determined by triangulation. For convenience in setting up and checking the Raydist equipment, points were established near the water's edge at both ends of these ranges. During the initial use of the Raydist system, it was discovered that the measured distances could not be checked by the distances obtained with the electronic equipment. Therefore, tests were performed with Raydist to determine the following: (1) The lane length; (2) the effect on the lane length when the master station is close to the shore station; (3) the effect on measurements when the master station is close to bluffs; and (4) the error due to phasemeter lag.

The lane length was checked and determined by running along lines established parallel to the axis of Garrison Dam. These lines were well targeted and flagged at the ends for the boat operator to keep the boat on line. Three transits were set on 90° offsets from the crest movements on the axis of the dam. Whenever the boat passed the line of a transit, flag and radio signals were given for the Raydist operator to read and record the phasemeter reading. Several runs were made along each of the three lines parallel to the axis of the dam. The average of these runs revealed an average lane length of 102.2, which was quite different from the theoretical or computed value of 102.5 feet. The theoretical lane length can be computed by the equation $\lambda = \frac{c}{f}$; where λ is the wave length, c=the velocity of propagation of radio waves (299,794 km./sec.),[5] and f=the radio frequency (4,796.4 kc.) of the basic transmitter. If this equation is used, the computed lane length, $\lambda/2 = \frac{(c)}{2f}\frac{(3.2808)}{} = 102.53$ feet. This measured or actual lane length was later confirmed by numerous surveys of ranges. It is recognized that corrections for the various factors affecting the propagation of radio waves are necessary to obtain true distances; however, by direct calibration as described above the necessity for determining the various corrections was eliminated.

During our initial surveys with Raydist, there was an indication that the lane lengths were shorter when the master station came close to the shore station. To obtain information on this phenomena, stadia and Raydist readings were taken simultaneously, each 50 or 100 feet for 1,000 feet from the shore station. In general, it was discovered that it was necessary to separate the stations 600 feet before this error was eliminated and that most of the error disappeared beyond 400 feet. Closer than 200 feet resulted in substantial errors. Figure 10 shows this error plotted against distance

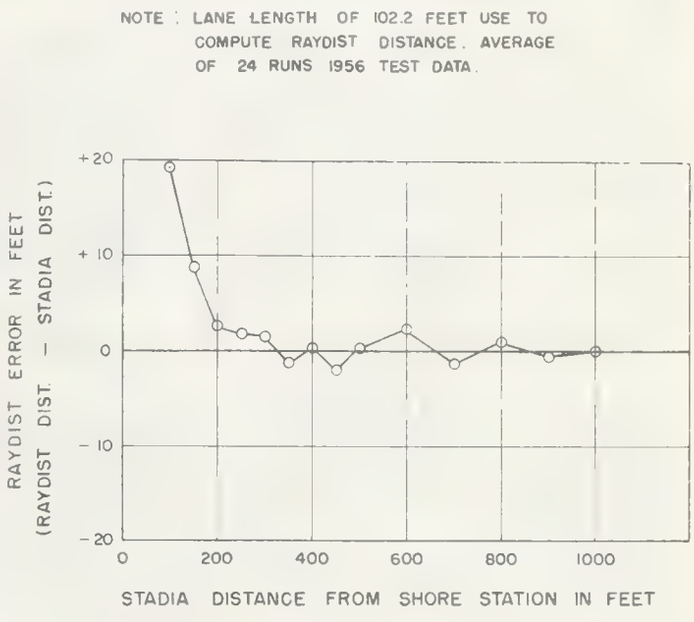


FIGURE 10. — Raydist error when master station is operating near shore station.

from the shore station. To overcome this, it has for the first 1,000 feet of soundings near the been necessary, in most instances, to use stadia shore station. Raydist readings are also obtained at each of these stadia fixes.

Also during some of the early surveys with Raydist, discrepancies were noted in the lane lengths obtained when the master station approached steep bluffs on that side of the reservoir opposite from the shore station. This error was investigated by simultaneously taking stadia and Raydist readings similar to that described above. Investigation was also made by towing the outboard cruiser with the shore station installed and operating. The master station and mobile transmitter were installed aboard the launch doing the towing. The cruiser motors were left down to cause some additional drag and thus increase the tension on the 350 feet of manila towline. The shore station was towed in the open reservoir until the phase meter revealed that the towline was adjusted, i.e., no change in the phase meter reading in-

[5] ASLAKSON, C. I. SOME ASPECTS OF ELECTRONIC SURVEYING. Amer. Soc. Civil Engin. Trans., 117 (Paper No. 2476): 1. 1952.

dicated that the length of the towline was not changing. The shore station was then towed past bluffs at varying angles and distances from the shore line. Any deviation of the phase meter now indicated reflection of the radio waves from these obstructions. It was necessary to stay about 300 feet from the bluffs to eliminate reflection. The maximum reflection observed when passing close to steep bluffs was about 0.20 lane. Some reflection was even observed when passing close to low banks approximately 10 feet high. This error is also eliminated by using both stadia and Raydist fixes on the first 1,000 feet of sounding near the opposite shore.

Slight errors due to phasemeter lag were observed during the initial use of Raydist. This error was investigated by taking Raydist readings while making backward and forward runs past the same locations. This error was found to be 0.03 to 0.05 lane. To eliminate this error, lane length ties for both ends of a range must be made with the master station launch pointed and moving in the same direction.

Preparatory Work for Aggradation Resurveys

Preparatory work applies to the survey work that is accomplished by a two- or three-man party prior to the actual sounding of the ranges and includes:

(1) Installation of the missing markers at the range monuments. The markers consist of 6- by 30-inch white boards fastened to steel fence posts.

(2) Establishment of control points near the water's edge for both elevation and stationing. If a large rise in the reservoir is forecast, additional control points are established at intervals of approximately 10 to 15 feet in elevation between the water surface and a point higher than that forecast. Flags are set at these control points.

(3) Profile the end of the range from the water's edge to a point beyond where no change has taken place.

(4) Determination of the water surface elevation. This elevation is checked against that obtained on the opposite shore and against that recorded by the lake gage. This work saves considerable time for the sounding party since it eliminates doing a substantial amount of survey work and searching for the ranges in cases where the markers have been damaged or destroyed.

Description of Sounding Procedure

Personnel Required

Four to eight personnel can be used to make up the hydrographic party depending upon the amount of preparatory survey work performed, the amount of sampling, etc., that is required in conjunction with sounding, and other workload items that affect the availability of personnel. Generally, six personnel are used — two at the shore station and four with the master station. The master station party includes a party chief, boat operator, Raydist operator, and sounder operator.

Setting Up the Shore Station

The most desirable end of the range is selected by the shore station party. A location near the range where the cruiser can be beached is preferred. The main duties of the shore station party are: (1) to set up and operate the shore station; (2) to profile the portion of the range between the water's edge and the nearest control point; (3) to run levels to determine the water surface elevation; and (4) to give alinement instructions to the operator of the sounding launch. After going ashore, the party performs the necessary survey work to establish the location for setting up the Raydist units. The units are set normal and about 17 feet on each side of the range line. The offset distance is usually determined by the lengths of the power cord from each cabinet which are plugged into the heavy duty power cord at the range line. The transit is usually set up at the point between the Raydist units for reading stadia and giving alinement instructions to the sounding launch. A portable "handi-talkie" radio is used to give alinement instructions to the boat operator. The shore station AC power supply, instead of the standard battery supply, is usually used for the radio. Figure 11 is a photograph of a typical setup of the shore station units.

FIGURE 11. — Typical arrangement of Raydist shore station units at Garrison Reservoir.

In instances where it is impossible or difficult to reach the shore because of shallow water, bluffs, or vertical banks, the Raydist units are installed on the cruiser. One unit is mounted on the bow while the other is mounted at the stern and the boat is secured across the range line so that the units are equidistant and normal to the line.

Positioning Raydist

It is necessary to tie-in the Raydist phasemeter readings to the range stationing at both ends of the range. When beginning or ending soundings on the *shore station side,* positioning of the Raydist is accomplished by —

(1) Repeated stadia position checks at 600 or 700 feet from the shore station.
(2) Chaining to the nearest control point when the Raydist shore station is set over 600 feet from the water's edge, except when shallow water prevents the sounding launch from coming close to shore, in which case it is necessary to use stadia.

When beginning or ending soundings on the *opposite shore side,* the positioning of Raydist is accomplished by —

(1) Chaining when there is deep water and no bluffs.
(2) A 600- or 700-foot stadia tie when there is shallow water or bluffs along the shore.

Stadia ties are made with the boat moving very slow and are repeated until two phasemeter readings agree with 0.02 or 0.03 lane length. More than likely, a check reading is also obtained during the sounding near shore when both stadia and Raydist are used.

Sounding of Long Deep Ranges

The shore station party travels to the most adaptable side of the reservoir for setting up the station. Meanwhile, the sounding launch performs a bar check at a deep water location near the range line. As soon as the shore station party has completed setting up its Raydist units, the units are switched from "Off" to "Standby" and the party proceeds with other duties. Personnel on the sounding launch turn on their Raydist units during the bar check so that there is a sufficient (at least 5 minutes) warmup period prior to using the equipment. After completion of the bar check, the shore party personnel switch their Raydist units from "Standby" to "On" and tune the two transmitters. At the same time, personnel on the sounding launch switch the mobile transmitter from "Standby" to "On," tune the transmitter, and check the operation of the entire Raydist system by observing the phase meter and the test set.

When the sounding operation is started from the opposite shore, two personnel and necessary equipment are taken to shore by either the sounding launch or in the case of shallow water by the small boat. These personnel then perform the necessary shore work, position the sounding launch, and if required give stadia fixes to the launch for the 1,000 feet of sounding near shore. One man is left on shore to give alinement to the sounding launch during the main sounding run.

When the sounding is started from the shore station side, the 1,000-foot portion of sounding is accomplished while traveling toward shore; after which, the positioning is accomplished while leaving the shore. At this time, it is necessary for one man from either the sounding launch or the shore station to go to the opposite shore in the small boat in order that he can be setup before the sounding launch arrives. One man must remain on the shore to give alinement to the sounding launch.

The main sounding run (from either shore) is started within the 1,000-foot portion by turning on the depth recorder and the "automatic fix." Position lines are then automatically recorded on the sounding chart for each even lane. The sounding, positioning, etc., at the other end of the range is accomplished in a similar manner to that previously described. Alinement instructions are given the launch operator by radio from either shore. Generally, and sometimes because of radio transmission difficulties, alinement is given for about one-half the range by each transit operator. Also, if some shoreline work remains on either side, the transit operator on the other shore takes care of the alinement duties.

Sounding of Long Ranges with Shallow Spots

Usually on every survey a few ranges are encountered that have areas shallower than the draft of the sounding launch. On Garrison Reservoir these situations are eliminated or at least reduced by surveying as many as possible of the ranges in the backwater reach during low reservoir stages prior to the June rise. If there have been changes (usually deposition) in the overbank, this part of the range profile is later obtained by conventional survey methods. When the launch cannot be used, the mobile transmitter is mounted in one of the smaller boats to obtain the shallow water profile of these ranges. The shore station is set up at one end of the range and the launch containing the master station is anchored at the opposite end of that part of the range to be sounded. Positioning is accomplished in the same manner as previously described. Fixes for the sounding chart are transmitted by radio

from the master station to the small sounding boat.

Use of Calculator

A Monro-Matic calculator is carried on the boat for determining lane lengths and range stationing. The range stationing is placed in the lower dial, the phase meter reading in the upper dial and the lane length in the keyboard. Range stationing for a particular phase meter reading can be obtained by operating the + and — keys so that this meter reading appears in the upper dial. A phase meter reading for a particular stationing can likewise be obtained. The range stations for the sounding "fixes" are determined in this manner and placed on the sounding charts during slack work periods; such as, when traveling between ranges and during periods when one of the master station crew is not busy at other duties. Computations of locations for sampling etc. are also determined with the calculator.

Determination of Lane Length

Lane lengths are determined by dividing the total known distance covered for any given run by the number of Raydist lanes measured for that known distance. Predetermined range stationing is necessary on each shore to provide an overall known distance. In order to eliminate accumulated error from using a predetermined lane length, a lane length is computed and used for each survey of a range. This eliminates prorating any error for the entire survey of a range.

Sampling Procedures

After sounding a range, locations are selected from the sounding chart for taking samples and using the sediment density probe. One observation in the channel and one or two on the overbank are usually adequate. Anchoring of the launch close to the point selected is accomplished by observing the phase meter. After anchoring, samples of the bottom material and density probe readings are obtained. The Foerst sampler is used when the bottom material is "soupy" and not too dense to penetrate. The denser material is sampled by use of a tube-type instrument, sometimes called a "reservoir" or "density" sampler. Density-probe readings are taken at vertical intervals to the penetration limit of the probe. Quite often the crew determines the weight and volume of the samples and saves a small portion in moisture-proof containers for laboratory-size analysis and moisture content determination. On the launch the tube sampler is operated from the high starboard boom, the density probe from the high port boom, the Foerst sampler from a bow boom, and if other samplers, such as a bed sampler are required, from the other bow boom. At the deepest vertical, temperatures and Foerst samples are quite often taken at intervals of 5 or 10 feet. The temperatures are taken with a resistance-type thermometer to obtain information on the thermal stratification of the reservoir. In general, however, stratification data are obtained during a continuous run up or down the reservoir, so that the information is collected during a shorter time period.

Disadvantages and Limitations of Raydist

The limitations of this equipment must be acknowledged, so that procedures can be devised and prepared to overcome these deficiencies during work in the field. It is recognized that better accuracy would be obtained if we could eliminate the stadia ties when the master station is close to the shore station or bluffs. This could be accomplished by (1) spending additional time to carry the shore station units, motor generator, etc., a sufficient distance inland; and (2) by investigating to discover more about this error and how to adjust for it. Nevertheless, we feel that the accuracy obtained with the use of good stadia ties is satisfactory.

Very few failures of the equipment have occurred during a sounding run; more frequently, trouble is discovered prior to starting the run. Failures during field operations have been minimized by (1) thoroughly checking, several weeks prior to use, the equipment in the shop by a radio technician; i.e., all the tubes are checked and as many as possible of the individual components are checked; (2) having the entire system operated, tested, and adjusted by the radio technician; (3) keeping the motor generator units in good operating condition; and (4) carrying a tube checker, VTM, and a supply of tubes and other spare parts on the sounding launch. Personnel in the master station party are usually experienced in using the testing instruments; therefore, breakdowns can often be repaired in the field without calling out a radio technician.

Cost of Raydist Equipment

At the time of purchase in 1956, a complete type ER Raydist system cost $42,000. The cost of our single-dimension portion of a type ER Raydist system was about $27,000.

Since purchase of the Raydist system, Hastings-Raydist, Inc. has developed the type DM Raydist system, which is transistorized to a large extent; therefore, the units are smaller and lighter and require less power. As a result, the shore station can be operated entirely from batteries. This new system measures distance

directly in both dimensions; i.e., to plot position it is only necessary to swing circular arcs using the shore station points as centers.

Conclusions

Various methods and means can be utilized in maintaining horizontal control for the sedimentation survey of reservoirs and each procedure has its own particular advantages or disadvantages. Use of the Raydist-type system of control, however, offers several unique and distinct benefits, which can be advantageously used on almost any sizable reservoir; but, in particular, apply to those larger reservoirs of exceptional width. These advantages include (1) a continuous measurement of varying distances by an accurate, rapid, and convenient method; (2) ability to reposition quickly at any desired location for additional observations; (3) mobility and versatility of uses; and (4) elimination of a costly triangulation network assuming control by intersection. Disadvantages include high initial cost, some limitations when used near steep bluffs or for the measurement of small distances, and, to a minor degree, the need for the occasional assistance of a trained electronics technician.

The single dimension-type arrangement for Raydist operation is practical, and its characteristics and limitations have been adequately defined through a variety of specific field tests. We were naturally disappointed in discovering certain limitations but are now confident that through our established operational procedures, better than third-order accuracy can be maintained. The practical justification for obtaining this equipment has been well satisfied.

FORECASTING SEDIMENT DISTRIBUTION IN A RESERVOIR BY ELECTRONIC COMPUTER

[Paper No. 87]

By FORREST B. MORROW, *assistant chief, Sedimentation Unit, Hydrology Section, U. S. Army Engineer District, Fort Worth*

Synopsis

Considerable advancement has been made in the past two decades in the field of sedimentation. Basic sediment data have been accumulated by Federal and State agencies in the collection of samples, resurvey of reservoirs, and experimentation in laboratories. Theoretical analyses have been expanded, and the methods of collection, analysis, and interpretation of field data have been improved. With the advent of electronic data processing equipment it became apparent that the long, tedious computations that absorbed many man-hours of work could be programmed for computer analysis and could be accomplished in a fraction of the time previously required. A method for distributing sediment in reservoirs has been developed and programmed for the Model G-15D Bendix general purpose digital computer.

Introduction

It was realized many years ago that sediment would be deposited in a reservoir upstream from any structure that would impede the flow of the sediment-bearing stream upon which it was built. As the streamflow enters the reservoir, the velocity decreases due to the larger cross-sectional area and the larger sizes of sediment begin to settle out. Originally it was assumed that most of the sediment carried into a reservoir would eventually be deposited only in the headwaters area and at the bottom in the main or deepest part of the reservoir. Resurveys of reservoirs have proved this assumption erroneous and that sediment deposition occurs in various degrees throughout all reaches of a reservoir. The problem then became one of anticipating the sediment distribution at the various levels in a reservoir. Numerous approaches have been made to establish this distribution. As more information became available, the complexity of the influences affecting sediment deposition in a reservoir became more pronounced. The shape of the reservoir, the sediment characteristics, the regimen of inflow, the anticipated reservoir regulation, and the sediment-reservoir volume ratio must be considered each in its proper perspective in the selection of a procedure for predicting sediment distribution in a reservoir.

On studies in connection with investigations for space for sediment deposition in reservoirs within the Fort Worth District it is desirable to know the general trend of the ultimate distribution patterns of this sediment. Various approaches and procedures have been taken in developing methods of predicting the sediment deposition patterns. The method most recently adopted in the Fort Worth District is the empirical area reduction method as presented by Whitney M. Borland and Carl R. Miller.[1] This method was programmed for use on the Bendix G–15D digital computer.

Procedure

In recent years sediment distribution patterns were required in the Fort Worth District for a large number of reservoirs varying in size

and shapes for the various river basins. These patterns were studied in connection with investigations for the U.S. Study Commission — Texas, and for survey reports and detailed plans for construction of numerous Corps of Engineers reservoirs in the district. These studies were time consuming because they also involved investigations of numerous combinations or systems of the already large number of reservoirs. In order to permit meeting schedules, various procedures for estimating distribution of the sediment in reservoirs were investigated. The method described by Borland and Miller [1] was found to be readily adaptable to the various reservoir types in the river basins under investigation.

This procedure requires first that the total amount of sediment anticipated to be deposited in each reservoir be determined and that the reservoir be classified in accordance with the types established. In accordance with the distribution pattern for the type of reservoir under consideration, the elevation of the new or ultimate "zero storage" capacity after the deposition of the estimated volume of sediment is then determined by the mathematical process described in the referenced paper.[1]

The amount or quantity of anticipated sediment to be deposited in a reservoir is currently estimated by utilizing procedures recommended by the Texas Water Commission (formerly Texas Board of Water Engineers) and the Soil Conservation Service.[2] The average annual sediment production rate for the watershed above a given reservoir is determined from the various land resource areas and their appropriate unit rates of sediment yield as established in the publication. The weighted average annual rate of sediment production is then applied to the number of years for which sediment deposition storage space is provided in the project. The elevation of the new zero storage point is then approximated by successive steps through trial and error. This latter procedure is time consuming.

The type of reservoir or curve used to distribute this sediment is established by utilizing the standard type curves presented by Borland and Miller [3] as a basis. These curves represent the percentage of sediment deposition versus the relative reservoir depth between the streambed at the damsite and elevation of the reservoir at the maximum controlled storage. The type of curve that fits the reservoir involved is obtained by relating depth versus initial reservoir capacity with the standard type curves given in the paper. The curves may then be adjusted for anticipated reservoir operation, type of sediment inflow, or capacity/inflow ratio. A specific curve of percentage of reservoir depth versus percentage of sediment deposited (sediment distribution curve) is finally adopted to satisfy the classification of the project under consideration.

Since the last step in this method, that of distributing the sediment, follows mathematical procedures, it is especially adaptable to determination by electronic computer. As presently programmed, the Bendix G–15D computer can determine the elevation of the new zero storage point, distribute the predetermined amount of sediment, and print out the original and revised area and capacity tabulations in a period varying from 20 to 30 minutes, depending upon the number of points in the original area and capacity tabulation and the precision required. The same computations if done manually would generally require from 4 to 8 hours.

The basic data required for the determination of the distribution of sediment in a reservoir by the adopted procedures consist of the original area and capacity tabulation, the amount of sediment to be distributed, the maximum reservoir level below which the sediment is to be distributed (usually top of controlled storage), the desired precision (usually 1 percent of the total amount of sediment to be distributed), and the reservoir type as classified above. The equation for standard type of distribution curves is part of the basic computer program. With these data the computer will determine by trial and error the following elements of the computations within the predetermined degree of accuracy desired:

(1) The revised area tabulation at successive increments of elevation.

(2) The revised capacity tabulation corresponding to the area data above.

At first an approximation of the elevation of the new zero storage point is determined by the computer using a trial and error procedure. With this approximate new zero elevation, the computer determines a revised area-capacity table and compares it with the amount of sediment to be distributed for the desired accuracy, repeating the computation until an elevation for the new zero storage point is obtained which is within the limits of desired precision. Once the final elevation of the new zero storage point has been determined, the computer prints out the following elements:

(1) Elevation;

(2) Original area;

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[1] BORLAND, W. M., and MILLER, C. R. DISTRIBUTION OF SEDIMENT IN LARGE RESERVOIRS. Amer. Soc. Civil Engin. Proc. 84, HY2 (Tech. Paper 1587). 1958.

[2] SOIL CONSERVATION SERVICE. INVENTORY AND USE OF SEDIMENTATION DATA IN TEXAS. Tex. Bd. Water Engin. Bul. 5912. 1959.

[3] See footnote 1.

(3) Original capacity;
(4) Revised area;
(5) Revised capacity.

In a basinwide investigation where numerous combinations and sizes of reservoir projects are studied and more than one distribution of sediment is made for a proposed reservoir, it is desirable to have the original and the revised area and capacity typed out on the same form for ease in identification. The program, as written for the Bendix G–15D computer, is limited to 50 elevation intervals. The elevation intervals may be set in either uniform or varying increments as desired, so long as the total number of intervals does not exceed 50.

The practice in the Fort Worth District has been to start at the topmost elevation of controlled storage and vary the intervals so that in the vicinity of the estimated elevation of the new zero storage point, the elevation interval is one foot. Once the new zero storage elevation has been determined and the computations completed for the initial 50 elevation intervals, the basic computation may be subdivided in order to obtain more detailed information at smaller intervals. During the early stages of its development, the Fort Worth Office of the Soil Conservation Service of the U.S. Department of Agriculture also utilized this program and facilities of the Corps of Engineers for estimating the distribution of sediment in reservoirs they were studying.

It is estimated that the current cost of operation of the Bendix G–15D computer based on approximately 100 percent utilization will not exceed $25 per hour.

Conclusion

The use of the Bendix G–15D computer for sediment distribution calculations has proved satisfactory in the Fort Worth District Office on the basis of man-hours saved in making the tedious computations and on the basis of making qualified engineers available for other studies or investigations that are not readily adaptable to computer programming. A review of an entire river basin can be made by the use of the computer in a fraction of time previously required for the manual distribution of sediment in the reservoirs. As the requirements arise, it is anticipated that many more sediment computations can and will be adapted to the computer programming.

DEVELOPING SEDIMENT STORAGE REQUIREMENTS FOR UPSTREAM RETARDING RESERVOIRS

[Paper No. 88]

By A. F. GEIGER, *geologist, Engineering and Watershed Planning Unit, Soil Conservation Service, U.S. Department of Agriculture, Milwaukee*

Abstract

Problems that are peculiar to the development of sediment storage requirements for upstream retarding reservoirs are discussed.

The wide variation in soil, topographic, and cover conditions that are possible between small watersheds, even within a physiographic region or major land resource area, leads to extreme variations in sediment yields. Measured sediment yield data for a specific watershed are usually lacking, and the time involved between planning and construction is too short to allow for the collection of meaningful data. Where sediment yield data are available, it must be modified to reflect the influence of the application of soil conservation measures.

To meet the specific policy requirements of the Soil Conservation Service pertaining to sediment capacity in retarding reservoirs, two methods are in general use. One involves the transposition of measured data from comparative watersheds. The second involves the estimation of watershed erosion and sediment delivery ratios, used when transposable data are unavailable.

Allocation of storage capacity for sediment accumulation within the reservoir basin requires consideration of the trap-efficiency, specific weight of sediment, and distribution of sediment within the basin. These considerations are discussed briefly.

General

Floodwater retarding reservoirs in small watersheds are designed to provide full effectiveness for a minimum period of 50 years. To assure the full effectiveness for the designed economic life, additional capacity must be provided in the reservoir to offset the depletion of capacity due to the accumulation of sediment within the reservoir.

The volume of storage that must be allocated to sediment is dependent upon the sediment yield from the watershed, the trap efficiency of the reservoir, the volume-weight of the deposited sediment, and, to a certain extent, the distribution of sediment within the basin. The most important of these is sediment yield.

Sediment yield can be defined as the total

amount of sediment that is delivered to or transported past a given point in a watershed. It is made up of that part of the products of erosion within the watershed which escape redeposition upstream from the point of measurement. In other words, sediment yield is dependent upon the total amount of erosion within the watershed (gross erosion) and the proportion of the products of erosion that is delivered to the point of measurement (delivery ratio). Delivery ratios are usually described as percentage of gross erosion. Since sediment is the product of erosion, factors that influence erosion must necessarily influence sediment yields. There is no direct relationship, however. One watershed may have a high rate of erosion but a low delivery rate, whereas another, with a relatively low ratio of erosion, may have a high delivery ratio.

The trap efficiency of a given reservoir is the ability of that reservoir to retain sediment which is delivered to it. Trap efficiency is the percentage relationship between sediment yield to a reservoir and sediment retained by the reservoir. It is dependent upon numerous factors, such as detention storage time, sediment particle characteristics, type of spillways and inlets, reservoir shape, and quality of water.

The volume-weight (dry weight) of sediment deposited within a reservoir may vary from less than 30 pounds per cubic foot to over 100 pounds per cubic foot. An overall average for measured volume-weights in existing small reservoirs would be in the neighborhood of 60 to 65 pounds per cubic foot. The ultimate volume that a given weight of sediment will occupy within a reservoir will depend upon its texture and mineral composition and whether it is permanently submerged or is alternately submerged and aerated.

The distribution of sediment in a reservoir is dependent upon the physical characteristics of the sediment, the shape and underwater topography of the reservoir, the nature of the channel above the reservoir, and the time required to empty the retarding pool. Capacity must be provided for all sediment that is deposited below the elevation of the emergency spillway.

Sediment Yields

At present the best known method of determining sediment yield is the long-term sampling of suspended sediment and bedload and recording of rainfall and runoff data on the watershed. Land-use data taken during the period of record may indicate the effects of land-use changes on sediment yield. These data combined with expected future land-use changes, can then be used to make a reasonable prediction of future sediment yields. Unfortunately such data are practically never available for small watersheds where structures are being planned. On occasion, an adjacent watershed having similar physical characteristics may have sediment yield data available. These data can often be transposed to the watershed in question, making adjustments for differences in drainage area and land use.

The most substantial source of measured sediment yield data comes from sedimentation surveys of existing small reservoirs. This provides a composite record of sediment yield from the drainage area above the reservoir for the period since the reservoir was built. However, it is usually impossible to isolate the effects of individual runoff events or to determine the effects of land-use changes. Periodic resurveys can sometimes overcome this difficulty.

The third alternative, when neither sediment load data nor reservoir survey data are available, is to determine rates of gross erosion and sediment delivery ratios. This involves the identification of the sources of sediment, computation of the volume of sediment available from each source, and an analysis of the factors that influence delivery rates.

Sediment Sources

A sediment source study is made to locate the origin of the sediment, to determine what part of the sediment is derived from each source, and to determine the effects of land treatment upon the production of sediment. The primary sources of sediment are sheet erosion, which is the removal of soil by the combined effects of raindrop impact and shallow overland flow, and channel erosion, which is the removal of soil by concentrated flow. Channel erosion includes gullying, streambed and bank erosion, and flood plain scour. There may be other sources of sediment within a local watershed, such as gravel washing plants or quarries.

Sheet erosion

Sheet erosion implies the removal of soil material from the land surface by overland flow. It is initiated by the impact energy of raindrops. Qualitatively, the primary factors influencing the rate of sheet erosion are, as summarized by Musgrave:[1]

(1) Rainfall, characterized particularly by intensities and amounts in the determination of the energy of impact;

(2) Flow characteristics of surface runoff, particularly as affected by slope degree and slope length;

(3) Soil characteristics, particularly those physical properties that affect erodibility; and

[1] MUSGRAVE, G. W. THE QUANTITATIVE EVALUATION OF FACTORS IN WATER EROSION — A FIRST APPROXIMATION. Soil and Water Conserv. Jour. 2 (3): 133–138. 1947.

(4) Vegetal cover, characterized by comparative densities and protective effects.

Data that bear upon the relation of these major factors to soil erosion were assembled from approximately 40,000 storms on fractional acre plots at 20 erosion experiment stations. A group of U.S. Soil Conservation Service research specialists was brought together to analyze these data. The results of the analysis were reported by Musgrave.[1] From the analysis the following equation was developed, which can be used for estimating the rate of sheet erosion under variable conditions:

$$E = IRS^{1.35} L^{0.35} P^{1.75}$$

where

E=soil loss, in inches;
I=inherent erodibility of the soil, in inches;
R=a factor describing cover conditions;
S=degree of slope, in percent;
L=length of slope, in feet;
P=maximum 30-minute intensity rainfall of a 2-year frequency, in inches.

The general procedure of the use of this empirical equation has been outlined by Gottschalk.[2]

More recently, the data used by Musgrave and additional data that have become available since the original analysis were subjected to another analysis. The result was the development of an improved soil loss equation. The general procedures for the use of this equation, which relate the same major factors to rates of soil erosion, have been outlined by Wischmeier and Smith.[3]

In both equations the vegetal cover and the use of conservation practices are shown to have a most profound effect upon land erosion. The rate of sheet erosion for clean-tilled crop or fallow land will be upwards of one hundred times the rate of erosion for established grass meadow, all other factors being equal.

Public Law 566 requires that not less than 50 percent of the lands situated in the drainage area above each retention reservoir must be under agreement to carry out recommended soil conservation measures and proper farm plans. In addition, not less than 75 percent of the effective land treatment must be installed, or their installation provided for, on those sediment source areas which, if uncontrolled, would require a material increase in the cost of construction, operation, or maintenance of the structural measure. These additional requirements must be scheduled for installation either before or concurrently with the installation of the structural measures.

This, then, necessitates the computation of gross erosion rates under "present" conditions, while soil conservation measures are being applied or are becoming effective, and under "future" conditions, when the soil conservation measures have become fully effective.

The average annual rate of sheet erosion under present conditions is computed from existing soil-survey and land-use data, supplemented by field reconnaissance where necessary. The total rate of sheet erosion on a watershed is determined by the summation of the sheet erosion on individual, more or less homogeneous increments. This is the rate that is used until the end of the project period, at which time all required land-treatment measures will have been applied and become effective.

The average annual rate of sheet erosion which will prevail after the installation of required land-treatment measures is also computed. This computation takes into account the effectiveness of better crop rotations, stripcropping, stubble mulching, contouring, terracing, land conversion, and controlled grazing in the control of erosion. This is the rate that is used for the remainder of the project period. The Department of Agriculture, in cooperation with the local sponsoring organization, has the responsibility of determining what these required land-treatment measures are, and the local sponsoring organization has the responsibility of applying the measures to the land.

Channel erosion

Channel erosion implies erosion by concentrated flow and is influenced mainly by such factors as channel gradient, channel soil characteristics, hydraulic parameters, and sediment supply and characteristics. In channel stability studies, these factors must all be considered. However, existing criteria are inadequate for estimating the expected volume of channel erosion under the highly variable conditions which occur within a watershed. Consequently more generalized methods have been developed by the Soil Conservation Service. This involves the field measurement of the net enlargement of channels over a known period of time, the time factor being determined through the use of aerial photographs of different ages and by interview of the local people. The average annual rate of channel erosion is then applied to the project period.

Other sediment sources

Other sources of sediment that occur within a watershed must be studied individually and control methods evaluated. The effects of urban-

[2] GOTTSCHALK, L. C. PREDICTING EROSION AND SEDIMENT YIELDS. Internatl. Union Geodesy and Geophys., Gen. Assembly 11, 1 (tome 1): 269-276. 1958.

[3] WISCHMEIER, W. H., and SMITH, D.D. SOIL-LOSS ESTIMATION AS A TOOL IN SOIL AND WATER MANAGEMENT PLANNING. Internatl. Assoc. Sci. Hydrology, Comm. on Land Erosion Pub. 59, pp. 148-159. 1962.

ization on sedimentation rates are of special interest. Only limited studies have been conducted to date, but these studies indicate the seriousness of the problem brought on by the rapid expansion of urban construction.

Sediment Delivery Ratios

Ideally, the delivery ratios of sediment from each of the various sources of erosion should be determined and applied to compute sediment yields. However, determining the source of sediment after it reaches the point of measurement (sediment samples of reservoir surveys) is usually impossible, and delivery ratios must therefore be determined on an overall basis.

The determination of sediment delivery ratios when both sediment yield and gross erosion are known is a simple matter of arithmetic. When sediment yields are not known and only gross erosion can be computed, it is necessary to estimate delivery ratios so that sediment storage requirements can be computed.

The relationship of sediment delivery ratios to measurable influencing factors, such as the magnitude of sediment sources, climate, soils characteristics, topography and other watershed characteristics, and depositional environments, has been analyzed by several workers in the field of sedimentation. The results of several of these analyses have been reported by Roehl.[4] Significant relations appear to exist between delivery ratios and area of drainage, average total stream length, and relief-length ratio. Local and regional analyses have been made, and curves have been developed in most areas of the country which relate delivery ratios to these physical factors. These curves are used locally in the estimation of expected future sediment yields from small watersheds.

Trap Efficiency of Reservoirs

The trap efficiency of a reservoir is the percentage relation between the amount of sediment that is delivered to a reservoir and the amount that remains within the reservoir. It is a function of detention storage time, character of sediment, type and location of spillways, reservoir shape, quality of water, and probably several other factors. An analysis conducted by Brune [5] indicates a close relation between trap efficiency and the ratio of the capacity of a reservoir to the average annual inflow. This relation, as used by the Soil Conservation Serv-

[5] BRUNE, G. M. TRAP EFFICIENCY OF RESERVOIRS. Amer. Geophys. Union Trans. 34 (3): 407-318. 1953.

[4] ROEHL, J. W. SEDIMENT SOURCE AREAS, DELIVERY RATIOS AND INFLUENCING MORPHOLOGICAL FACTORS. Internatl. Assoc. Sci. Hydrology. Comn. Land Erosion Pub. 59, pp. 202-213. 1962.

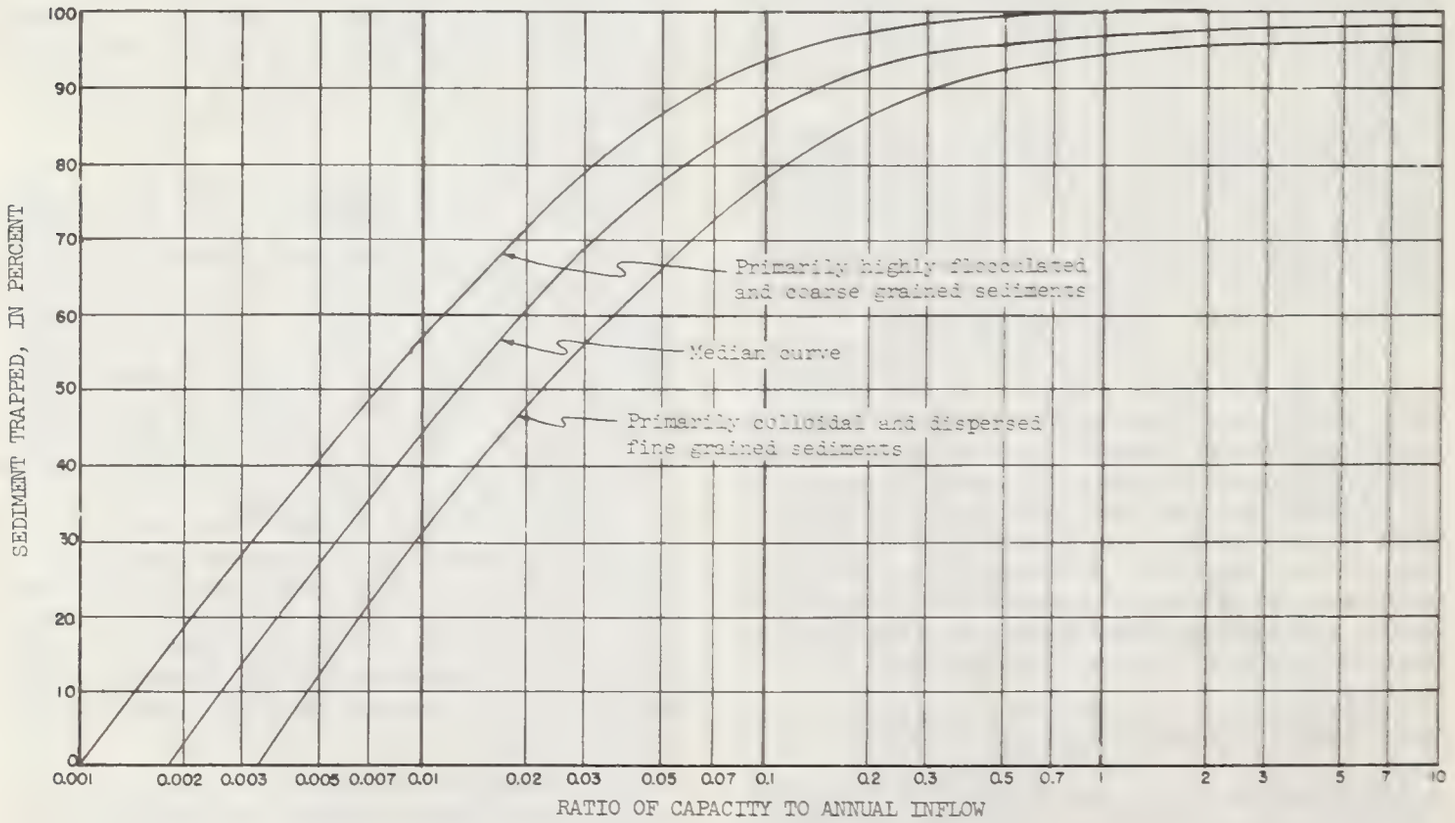


FIGURE 1. — Trap efficiency of reservoirs. (Adapted from Brune.)

ice, is illustrated by figure 1. These curves allow for the modification of trap efficiency estimates based on the expected character of sediment. Capacity inflow ratios in small detention reservoirs are seldom less than 0.1 and trap efficiencies are seldom less than 90 percent.

Volume Weight of Sediment

The ultimate volume occupied by deposited sediment will depend essentially upon its texture and whether it is permanently submerged or is subject to alternate submergence and aeration. Where possible, volume-weight analyses are made of sediments in existing reservoirs to aid in the prediction of volume weights in proposed reservoirs. Because it is not always possible to make field measurements, table 1 has been devised, based on the analysis of available data on mechanical analyses and corresponding volume weights of sediments.

TABLE 1.—*Indicated ranges in volume-weight of reservoir sediment*

Dominant grain size	Permanently submerged	Aerated
	Lb./cu. ft.	*Lb./cu. ft.*
Clay	40 to 60	60 to 80.
Silt	55 to 75	75 to 85.
Clay–silt mixture	40 to 65	65 to 85.
Sand–silt mixture	75 to 95	95 to 110.
Clay–silt–sand mixture	50 to 80	80 to 100.
Sand	85 to 100	85 to 100.
Gravel	85 to 125	85 to 125.
Poorly sorted sand and gravel	95 to 130	95 to 130.

With a knowledge of the character of the inflowing sediment and whether it will be submerged or aerated, an acceptable estimate of the volume weight of sediment in a proposed reservoir can be made with the aid of the table. Since soil in place will have a different volume than the same weight of soil deposited as sediment, it is necessary to make all estimates of sediment yield as weights instead of volumes.

Sediment Distribution

The portion of the incoming sediment that will deposit above the elevation of the principal spillway will vary with the amount and character of the sediment and the stage of the reservoir. In general, a larger part of the coarse-grained sediment will be deposited above this elevation. The general distribution of sediment below the principal spillway, after its elevation has been set, is of little consequence. However, that portion that is deposited above the principal spillway will have a bearing on the elevation of the spillway. A given volume of storage is allocated to sediment. This volume, if all sediment would occupy the lowest parts of the pool, would normally determine the elevation of the principal spillway with the detention pool above this elevation. When a portion of the sediment is deposited in the detention pool, the elevation of the principal spillway must be lowered by an amount equal to this volume of storage.

Little information is available on sediment distribution in retarding reservoirs, but from an analysis of available data the following guide lines have been developed to aid in estimating deposition above the principal spillway.

(1) In watersheds of low to moderate relief where the predominant sources of sediment are silty and clayey soils, where sheet flow is the principal eroding agent, and the sediment is transported primarily in suspension, approximately 10 percent will be deposited above the principal spillway.

(2) In watersheds of low to moderate relief, with combination sheet and channel erosion, where products of erosion are essentially equal amounts of medium to fine sands, silts and clays, transport of coarser materials is along the bed and finer materials are in suspension, approximately 20 percent will be deposited above the principal spillway.

(3) In watersheds of moderately high relief where the primary source of sediment is from channel erosion producing substantial amounts of coarse sand and gravel that are transported as bedload, with smaller proportion of fine-grained sediment transported in suspension, approximately 30 percent.

(4) In watersheds of high relief where the primary sediment load consists of boulders, cobbles, and sand, above 30 percent.

These percentages may be adjusted upward or downward, using judgment based on local watershed conditions.

These estimates are also used to help in determining what portion of the deposited sediment will be aerated and what portion will be permanently submerged. Other factors will also enter into this determination. For instance, drawdown outlets may be provided so that the sediment pool will be "dry." Or seepage and/or evaporation losses may exceed inflow for long periods of time.

Sediment Storage Requirements

The foregoing procedures and computations are used by the Soil Conservation Service in the determination of sediment storage requirements and the allocation of this volume to various portions of the reservoir. The Soil Conservation Service is continually striving to improve its estimates and to develop new tools and procedures. The collection and analysis of data that is becoming available as the small watershed program advances will aid in the refinement of estimates and improvement of procedures. Continuing research in the field of sedimentation is doing much to this end.

A NEW METHOD OF ESTIMATING DEBRIS-STORAGE REQUIREMENTS FOR DEBRIS BASINS

[Paper No. 89]

By FRED E. TATUM, *chief, Hydrology and Reservoir Regulations Section, U.S. Army Engineer District, Los Angeles*

Synopsis

A method was needed for computing the debris production for all areas for which debris basins are to be built. It is recognized that there are many factors affecting debris production. The method presented in this paper was based on observed data, most of which resulted from floods that occurred when the ground was conditioned for runoff by prior rain and from areas that had been partially or completely burned 1 to more than 10 years prior to the flood. The observed debris amounts were adjusted to a common base and curves were developed by trial and error to represent separate adjustments for the major factors affecting debris production. This procedure is then adapted to estimating debris production for areas where debris basins are to be built.

Introduction

The design debris capacity for debris basins in the Los Angeles area of the Los Angeles District originally was based on a flat rate of 50,000 to 100,000 cubic yards per square mile of drainage area, depending upon the results of an inspection of the area. Later, when more information had been compiled, the debris basin capacity was based on an enveloping curve of debris inflows plus 100 percent (fig. 1) and a field inspection of the drainage area. It was realized that a more detailed method of determining the debris capacity for use in the design of debris basins was required to afford equal protection to areas below debris basins. To meet this requirement a new method of computing debris production from a drainage area was developed. The method is based on evaluating the main factors that affect debris production. The development and use of the method are presented in this paper.

Available information on debris deposition at alluvial cones and on debris inflow at reservoirs and existing debris basins in the Los Angeles area, as well as in generally comparable areas in the Southwest, was evaluated. It was recognized that in the production of debris, several factors are involved, including the size and shape of the drainage area; the steepness of canyons and side slopes; geological characteristics (such as type of rock and soil, and weathering effects); the type and density of plant cover; recency of burns; and the frequency, the duration, and, in particular, the intensity of storms and floods.

Debris-Production Factors

A review was made of all available reports on debris, and after a study of the records of major debris flow in the Los Angeles County area, the factors that were found to have the most effect on debris production are as follows: (1) drainage area, (2) slope, (3) drainage density, (4) hypsometric-analysis index, (5) 3-hour rainfall, and (6) burn effect. Definition of the factors as used in this paper are given in the following paragraphs.

Drainage Area

The drainage area is the area upstream from the considered concentration point or debris-basin site, in square miles.

Slope

The slope is the average slope of the longest watercourse upstream from the considered concentration point or debris-basin site, in feet per mile.

Drainage Density

The drainage density is the total stream length in miles, divided by the drainage area in square miles. The total stream length is the summation of stream lengths of all streams indicated by those streamflow lines upstream from the considered concentration point or debris-basin site that are shown as either intermittent or perennial on the pertinent U.S. Geological Survey map. (1/24,000 scale U.S. Geological Survey maps were used in this study.)

Hypsometric-Analysis Index

The hypsometric-analysis index is based on the hypsometric analysis developed by W. B. Langbein and others and presented in U.S. Geological Survey Water Supply Paper 968–C, dated 1947. The analysis involves the development of a nondimensional elevation-area curve with unity equaling total height and total drainage area above the considered concentration point (fig. 2). Arthur N. Strahler, in an article titled "Quantitative Analysis of Watershed Geomorphology," published in the December 1957 "Transactions, American Geophysical Union," states that the hypsometric curve properties tend to adhere generally to the same curve family for a given geological and climatic combination.

The hypsometric-analysis index, as developed by the Los Angeles Engineer District, is the relative height at which the drainage area is divided into two equal parts (see fig. 2, relative

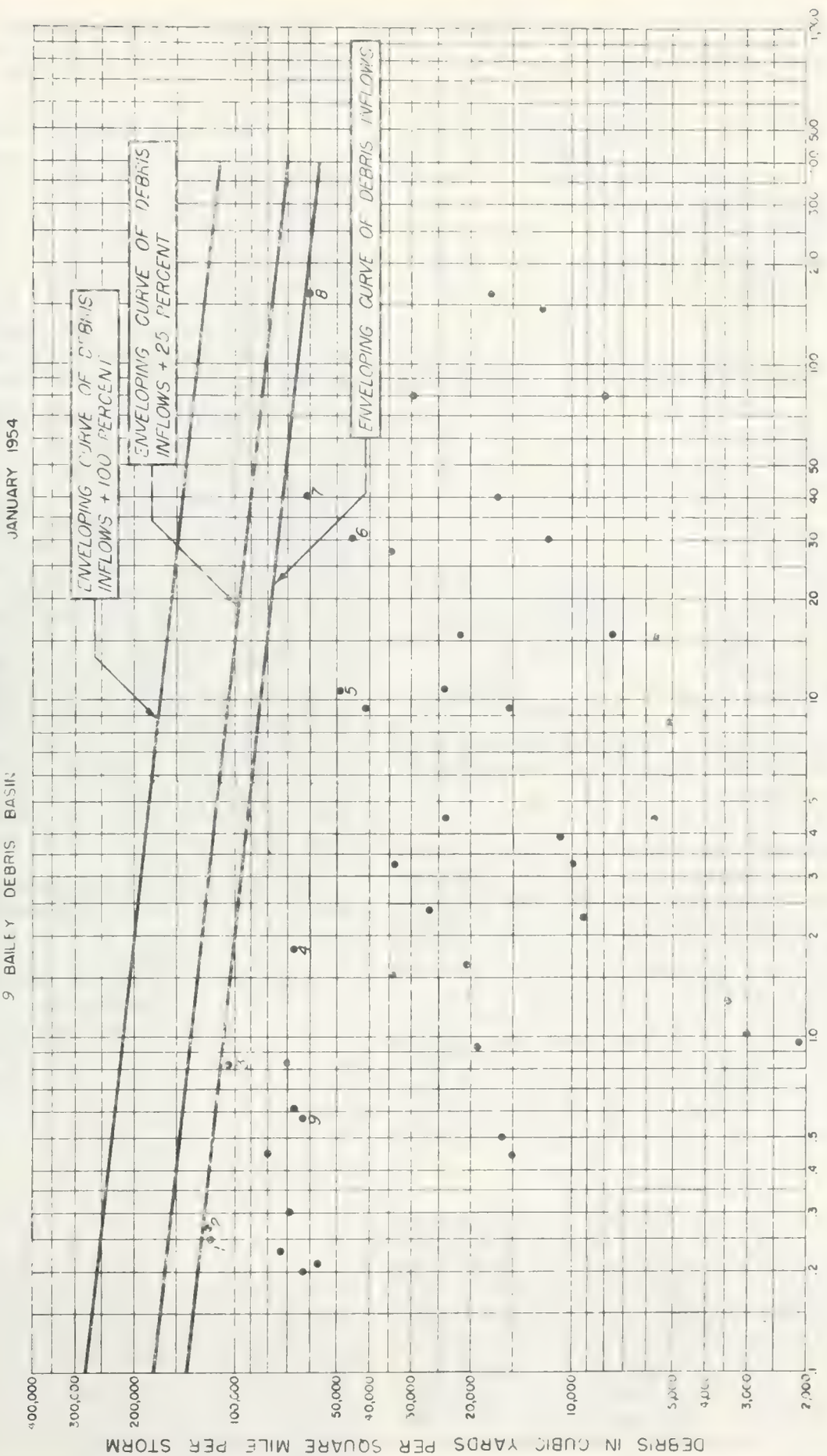

LEGEND
RECORDED OR ESTIMATED DEBRIS INFLOW
1 WEST RAVINE DEBRIS BASIN MARCH 1938
2 SHIELD DEBRIS BASIN MARCH 1938
3 HALL-BECKLEY DEBRIS BASIN MARCH 1938
4 PICKENS DEBRIS BASIN MARCH 1938
5 SANTA ANITA FLOOD-CONTROL BASIN MARCH 1938
6 DEVIL'S GATE FLOOD CONTROL BASIN MARCH 1938
7 COGSWELL FLOOD-CONTROL BASIN MARCH 1938
8 SAN GABRIEL FLOOD-CONTROL BASIN MARCH 1938
9 BAILEY DEBRIS BASIN JANUARY 1954
ENVELOPING CURVE OF DEBRIS INFLOWS + 100 PERCENT
ENVELOPING CURVE OF DEBRIS INFLOWS + 25 PERCENT
ENVELOPING CURVE OF DEBRIS INFLOWS
DEBRIS IN CUBIC YARDS PER SQUARE MILE PER STORM
DRAINAGE AREA IN SQUARE MILES

FIGURE 1. — Debris inflows, debris, and flood control basins in Los Angeles County, Calif

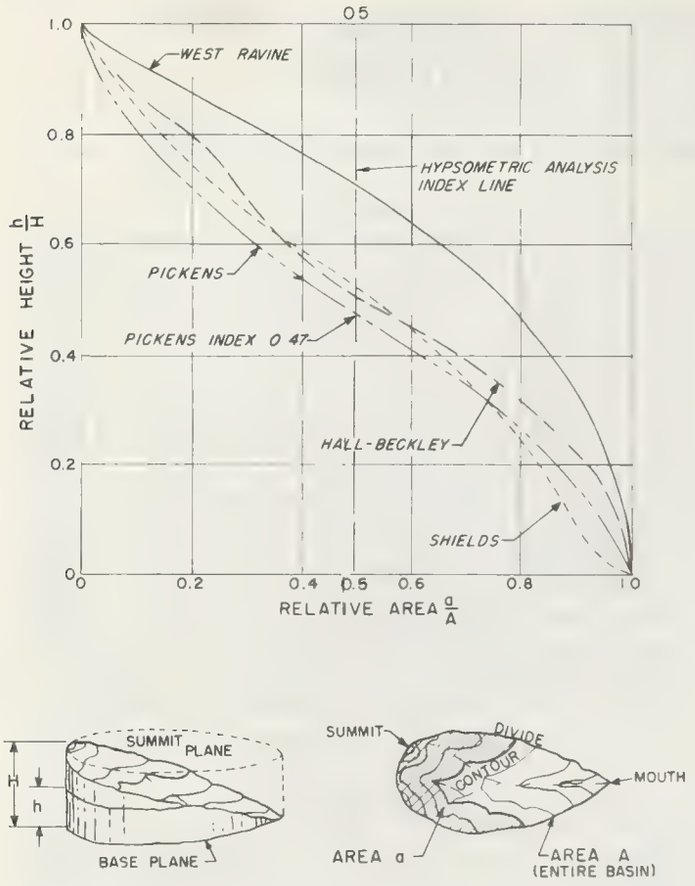


FIGURE 2. — Hypsometric curves for drainage areas above selected debris basins in Los Angeles County, Calif.

area equals 0.5). A comparison of the index numbers with the geology for the drainage areas studied indicated that soil most likely to erode was generally in drainage areas with the index number equal to or less than 0.5. At the same time, index numbers less than 0.5 reflect the lessened debris potential affected by the concentration points being farther removed from the steeper part of the drainage area. Index numbers larger than 0.5 indicate soils generally less likely to erode. The relation of the hypsometric-analysis index to an analysis of the drainage area by a geologist is shown in table 1.

Three-Hour Rainfall

The 3-hour rainfall in inches is the maximum 3-hour rainfall occurring during the storm after prior rain has conditioned the ground for debris movement. This 3-hour period of rain was determined to be the critical period of rainfall for use in determining the effect of rain on debris production. The area that produced most of the debris quantities used in this study during the 1938 flood were conditioned for debris production by rain prior to the maximum 3-hour rainfall.

Burn Effect

The burn effect depends on the percentage of drainage area burned and the recency of the burn.

Observed Debris Production

Data on debris flow have been obtained from (1) a paper by B. H. Dodge titled "Design and Operation of Debris Basin," published in proceedings of the Federal Interagency Sedimentation Conference, Denver, Colo., and dated January 1948; and (2) reports and unpublished data of Los Angeles County Flood Control District. Pertinent data and observed debris production are given in table 2 and the locations of the debris basins are shown on figure 3.

Adjusted Observed Data

To determine the effects of the various factors on production of debris, it was necessary to adjust the observed data to a common base. This was done by adjusting the observed debris production to the amount that would have been produced if the storm had occurred the first year after 100 percent burn in the drainage area. The adjustment was based on data from the report by California Forest and Range Experimental Station, Forest Service, U.S. Department of Agriculture, titled "Hydrologic Analysis Used to Determine Effects of Fire on Peak Discharge and Erosion Rates in Southern California Watersheds," by P. B. Rowe, C. M. Countryman, and H. C. Storey, and dated February 1954. Hereafter in this report, that report will be referred to as the U.S. Forest Service report.

The observed debris production from a drainage area (A) can be represented by the equation:

$$X_o = X_{10} A_n + X_v A_b \quad (1)$$

Where X_o = Observed debris production, in cubic yards;

X_{10} = Rate of debris production 10 or more years after burn, in cubic yards per square mile;

X_v = Rate of debris production for "v" year (1 to 10) after 100 percent burn, in cubic yards per square mile;

A_n = Portion of drainage area not burned, in square miles; and

A_b = Portion of drainage area burned, in square miles.

The relation of debris production rate in percentage of first year production and 100 percent burn to the debris production rate for "v" years (1 to 10) after the burn, based on data from the U.S. Forest Service report, is given on figure 4. The debris production rate for 10 years after a burn, in percentage of first year and 100 percent burn, is 3 percent. It is realized

TABLE 1.—*Relationship of the hypsometric-analysis index to the analysis of the areas by a geologist*

Debris basin or site	Drainage area	Hypsometric-analysis index	Geological analysis
	Square miles		
Brand	1.13	0.50	Brand Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.
Childs	.36	.46	Childs Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with a thin soil and a sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.
Sunset	1.11	.46	Sunset Canyon drainage basin is composed of closely fractured granitic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.
Elmwood	.34	.44	Elmwood Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with thin soil and a sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.
Blanchard	.50	.40	Blanchard Canyon drainage basin's geological factors affecting debris potential are comparable to those of nearby Haines Canyon; the debris potential is high.
Hillcrest	.41	.33	Hillcrest Canyon, and its right tributary (Sherer Canyon) in particular, has a drainage basin of closely fractured granitic and metamorphic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.
Wilson	2.54	0.55	Wilson Canyon drainage area is similar to that of other San Gabriel Mountains front canyons for about two-thirds of the drainage area and forms a definitely high potential for debris production. The debris potential of the rest of the area appears to be much less.
Schoolhouse	.28	.59	Schoolhouse Canyon drainage area is a definitely high potential for debris production from two-thirds of the area where the material is similar to other San Gabriel Mountains front canyons. The debris potential of the remaining one-third of the area overlying sedimentary rocks is only moderate.
Live Oak	.29	.65	Live Oak Canyon drainage area below Live Oak Dam is about 40 percent terrace area of gently rolling hills and about 60 percent more or less steep canyon slopes. The terrace area has a low debris potential, whereas the canyon portion has a moderate to high debris potential.
Emerald	.19	.65	Emerald Wash drainage area is about 75 percent terrace area with a low debris potential and about 25 percent steep canyon slopes with a moderate to high debris potential.
Emerald (East)	.53	.72	Emerald Wash (East) drainage area is about 70 percent terrace area with a low debris potential and about 30 percent steep canyon slopes with a moderate to high debris potential.
Lopez	1.56	.82	Lopez Canyon drainage area has a shallow soil cover and conditions are such that debris contribution should be substantially less than at areas along the mountain front to the east where debris production has been measured.

that the relationship developed in the U.S. Forest Service report was based on annual rainfall and may not be a true relationship for a single storm. However, this is a relative matter, in that all developed curves of separate adjustment factors are interrelated; therefore, a deviation from the relationship presented on figure 4 would tend to be compensated for in the development of the other curves. The relation of debris produced in a given year after a burn to the debris production percentage rate for that year is equal to the relationship for 10 years after, as in the following equation:

$$\frac{X_v}{C_v}=\frac{X_{10}}{3}$$

$$\text{or } X_v=\frac{C_v X_{10}}{3} \qquad (2)$$

FIGURE 3. — Location of debris basins.

TABLE 2.—*Observed data and computed debris production for selected debris basins in the Los Angeles area*[1]

Debris basin[2]			Burn in drainage area		Debris-producing flood		Observed debris production during flood		Observed debris rate adjusted to 100 percent burn 1st year[2]	Debris production factors for—				Correction factors					Computed debris production	
No.	Name	Drainage area	Year	Area	Year	Years after burn	Total	Rate		Slope	Drainage density	Hypsometric index	3 hour rainfall	Slope	Drainage density	Hypsometric index	3-hour rainfall	Total	For maximum 1 square mile with 100 percent burn 1st year[3]	For year of observed flood and actual area burned[2]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
	La Cresenta Area:	*Square miles*		*Square miles*			*Cubic yards*	*Cubic yards per square mile*	*Cubic yards per square mile*	*Feet per mile*	*Miles per square mile*		*Inches*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Cubic yards*	*Cubic yards*
1	Dunsmuir...	0.84	1933	0.78	1938	5	58,800	70,000	695,000	1,390	1.7	0.54	2.94	99	97	98	67	64	1,220,000	106,000
2	Eagle-Goss..	.61	1933	.46	1938	5	40,900	67,050	767,000	1,480	3.3	.25	2.89	100	84	93	64	18	342,000	19,400
3	Haines.....	1.53	1933	1.01	1938	5	52,000	33,990	423,000	1,040	2.4	.46	2.85	92	95	98	62	52	1,010,000	117,000
4	Hall-Beckley	.83	1933	.63	1938	5	86,300	103,980	1,185,000	930	2.2	.50	2.82	88	96	100	61	59	990,000	73,000
5	Hay	.20	1933	.06	1938	5	12,600	63,000	1,190,000	1,290	.6	.65	2.72	97	99	78	56	40	760,000	9,700
6	Pickens	1.84	1933	1.75	1938	5	122,200	66,410	650,000	940	3.4	.47	2.93	88	82	99	67	48	910,000	160,000
7	Shields.....	.27	1933	.24	1938	5	33,500	124,000	1,320,000	1,570	2.5	.51	2.90	100	94	100	65	61	1,260,000	34,400
8	Snover.....	.23	1933	.11	1938	5	16,800	73,040	1,110,000	1,280	3.5	.54	2.82	97	81	98	61	46	874,000	15,500
	Pasadena Area:																			
9	Fair Oaks...	.21	1935	.21	1938	3	12,000	57,140	257,000	1,180	0	.21	2.34	95	100	25	36	9	171,000	9,600
10	Fern.....	.30	1935	.30	1938	3	20,700	69,000	310,000	1,180	4.8	.41	2.42	95	51	92	39	17	323,000	24,800
11	Las Flores..	.45	1935	.31	1938	3	36,000	80,000	495,000	1,610	3.2	.58	2.62	100	85	94	49	39	741,000	60,000
12	Lincoln.....	.50	1935	.50	1938	3	8,000	16,000	72,000	780	4.6	.31	2.39	81	57	60	38	11	209,000	25,600
13	West Ravine	.25	1935	.24	1938	3	29,800	119,200	555,000	1,290	0	.40	2.40	97	100	90	38	33	627,000	39,500
14	Bailey.....	.58	1953	.46	1954	1	65,000	112,070	140,000	1,520	2.6	.50	1.70	100	93	100	12	11	209,000	104,000
	Burbank Area:																			
15	Brand.....	1.03	1927	.77	1943	[4]10	3,100	3,010	99,000	910	3.0	.50	1.70	87	88	100	12	9	171,000	5,300
16	Sunset.....	.44	1927	.42	1938	[4]10	6,600	15,000	495,000	1,610	2.0	.48	1.93	100	97	99	18	17	323,000	4,800
17	Stough.....	1.65	[4]	[4]	1943	[4]	33,500	20,300	670,000	1,020	4.4	.62	2.64	91	63	85	52	25	475,000	20,000
	Beverley Hills Area:																			
18	Nichols....	.94	[4]	[4]	1938	[4]	17,900	19,040	626,000	480	.9	.56	2.48	57	98	96	42	23	437,000	12,600

[1] See fig. 3 for location.
[2] Computed by use of equation 5 (see text) and adjusted for size of drainage area.
[3] 1,900,000 times total percent from column 19.
[4] 10 years or more assumed to have no effect on debris production.

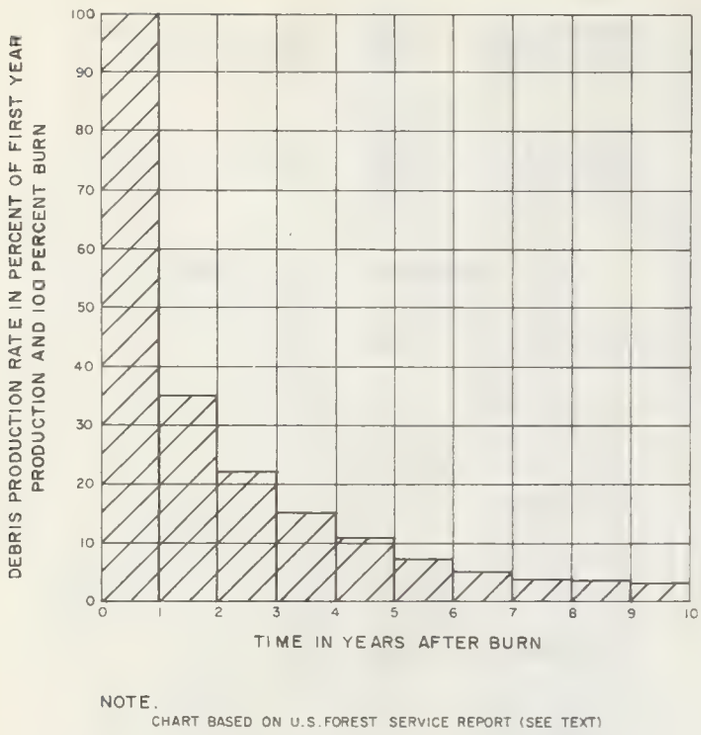


FIGURE 4. — Debris production rates following burn.

Where C_v=debris production rate for "v" year (1 to 10) after burn, in percentage of rate for first year and 100 percent burn.

Equation 2 is substituted in equation 1:

$$X_o = X_{10} A_n = \frac{C_v X_{10} A_b}{3}$$

$$\text{or } X_{10} = \frac{3X_o}{3A_n + C_v A_b} \quad (3)$$

Also from figure 4 we get the relation:

$$\frac{X_1}{100} = \frac{X_{10}}{3}$$

$$\text{or } X_1 = \frac{100}{3} X_{10} \quad (4)$$

Where X_1=rate of debris production first year after 100 percent burn, in cubic yards per square mile.

If equation 3 is substituted in equation 4:

$$X_1 = \frac{100X_o}{3A_n + C_v A_b} \quad (5)$$

By means of equation 5 the observed rates of debris production were adjusted to first year after 100 percent burn and are given in table 2, column 10. The adjusted values were then plotted versus drainage area in square miles (fig. 5). A curve was then drawn through the two highest points (parallel to the enveloping curves shown on figure 1, which reflects the effect of the size of drainage area on debris production) to represent the ultimate debris-potential index of production the first year after 100 percent burn for a 3-hour rain of approximately 2.9 inches. This curve represents maximum debris from the areas with the more severe debris producing characteristics.

Development of Adjustment Curves

The development of method was based on the assumption that the debris potential for an area was equal to the product of the ultimate debris-potential index and the percentage values for each of the four factors (slope, drainage density, hypsometric-analysis index, and 3-hour rainfall) representative of the area. It was considered that each factor, if not the ultimate for the area, would reduce the debris potential index by a percentage representing the difference between the ultimate and actual. Curves were drawn to represent the relationship for each of the four factors. These curves were then adjusted by trial and error to give the best results when computed data were compared to observed data. The adjusted curves are shown on figures 6 and 7.

The steps used in computing debris production, for comparison with the observed values, are as follows: (1) the slope, drainage density, hypsometric-analysis index, and 3-hour rainfall for the debris-producing storm are determined for each area; (2) the percentage correction for the above factors are determined from the applicable curves on figures 6 and 7; (3) the percentage corrections are multiplied together to give a total correction; (4) the total correction is multiplied by 1,900,000 cubic yards, which — as shown on figure 8 — represents the ultimate debris-potential index for 1 square mile in maximum-debris-producing areas, to give the maximum debris production in cubic yards from areas of 1 square mile; (5) the value thus computed for maximum debris production in cubic yards for areas of 1 square mile is then adjusted for size of area under consideration by plotting the computed value on figure 5, drawing a curve through the plotted point and parallel to the enveloping curve, and reading the value for the area under consideration from this line; and (6) the adjusted value is then corrected to the actual area burned and number of years after the burn by use of equation 5. The results thus obtained are given in table 2, column 21.

The comparison of the observed and computed debris production is shown on figure 9. The curve shown on figure 9 represents computed values equal to observed values and indicates that the relationship of computed to observed values are reasonable. The scatter of the points is partly due to (1) the difference in ground conditions prior to the flood, (2) the variation in ground cover, (3) the probable

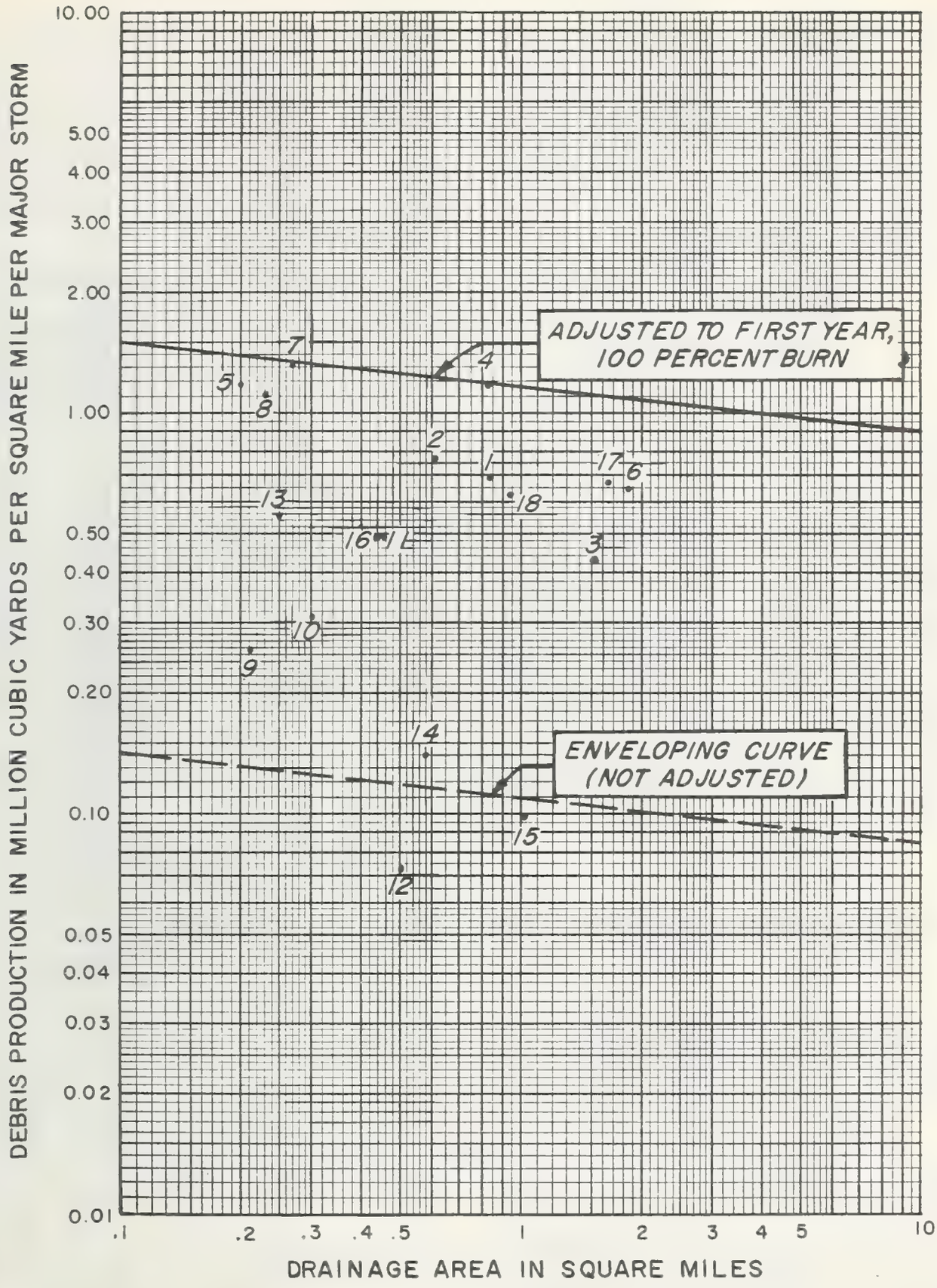


FIGURE 5. — Enveloping curves of observed debris production adjusted to 100 percent burn first year, for one major storm.

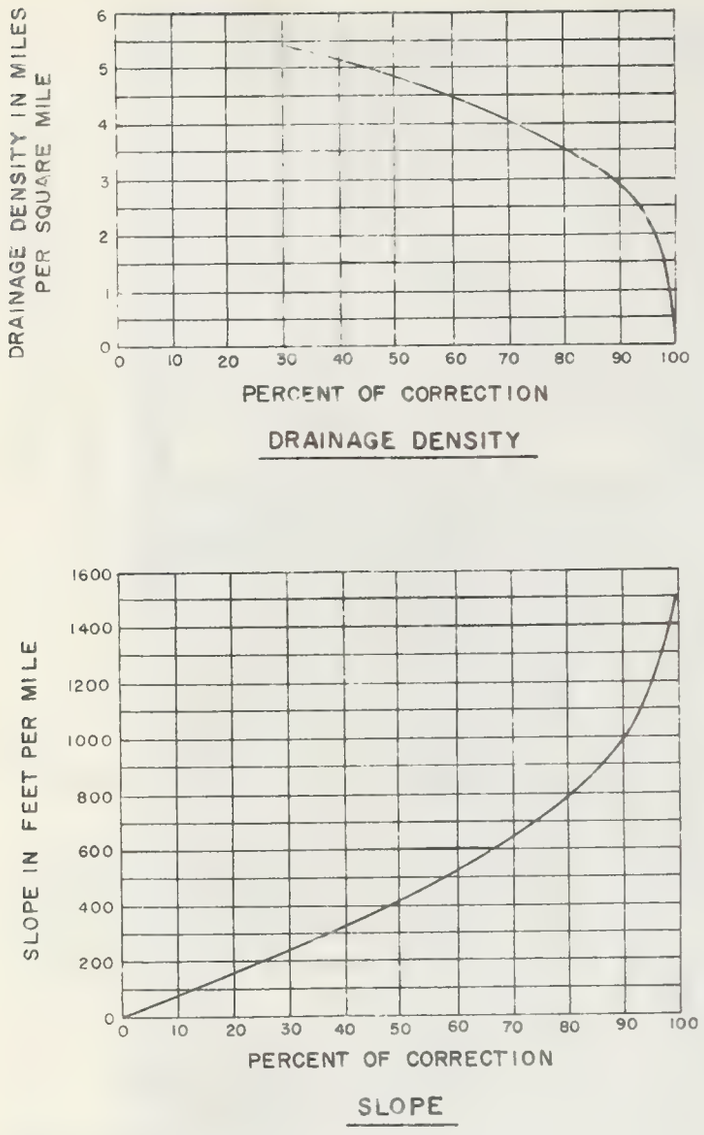


FIGURE 6. — Drainage density and slope curves indicating correction of debris production.

error in determining observed debris quantities and 3-hour rainfall amounts, and (4) the possible inconsistency in using the streamflow lines in determining total stream length to be used in the determination of drainage density.

Verification of Method

Verification of the method is indicated by comparing results obtained by computing the amount of debris produced during the storm of 7 to 12 February 1962 in the Los Angeles area, with amounts actually produced according to the Los Angeles County Flood Control District measurements. A fire occurred in the drainage areas of Sierra Madre Villa, Bailey Canyon, Auburn Canyon, and Carter Canyon debris basins (fig. 3) in October 1961. A storm occurred over these areas on 7 to 12 February 1962, with a total rainfall of about 11 inches. The maximum 3-hour rain occurred in the evening of the 10th. Although prior to this storm the ground conditions were relatively dry, the rain prior to the maximum 3-hour rain was reasonably adequate to condition the ground for runoff. Pertinent information for drainage areas above the debris basins and the computed and measured amounts of debris are given in table 3.

Recommended for Design

The design capacity of a debris basin should represent that amount of debris for one major storm that would be exceeded in magnitude only on rare occasions. The assumption that a major storm would occur, with ground conditions con-

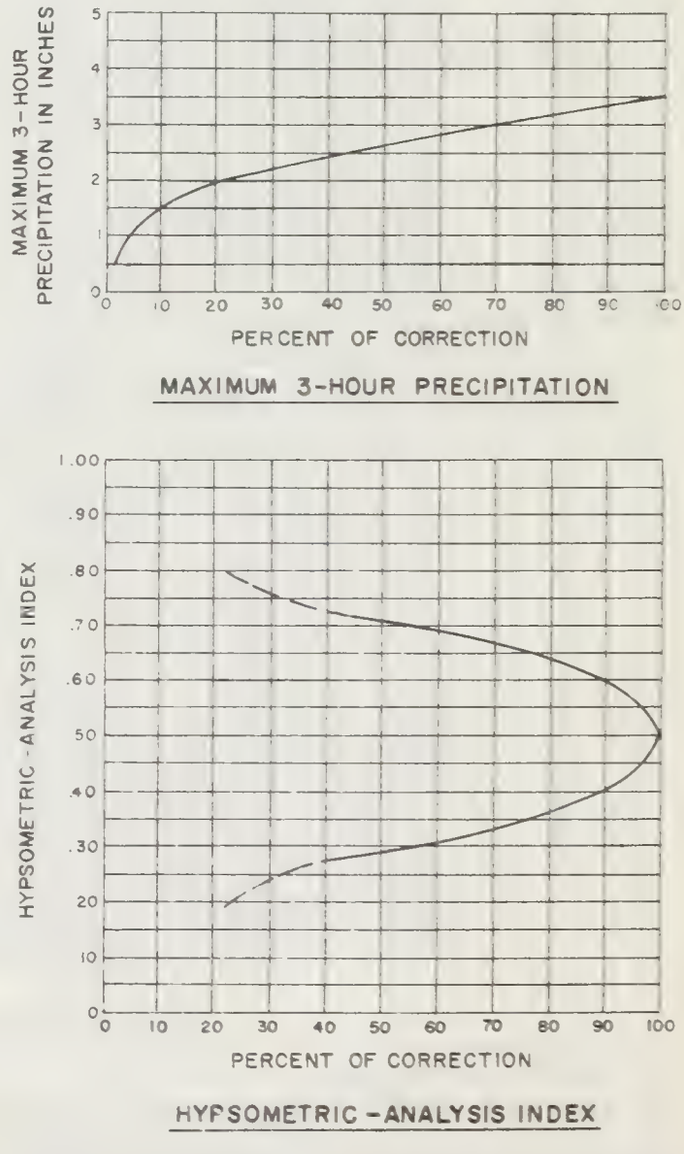
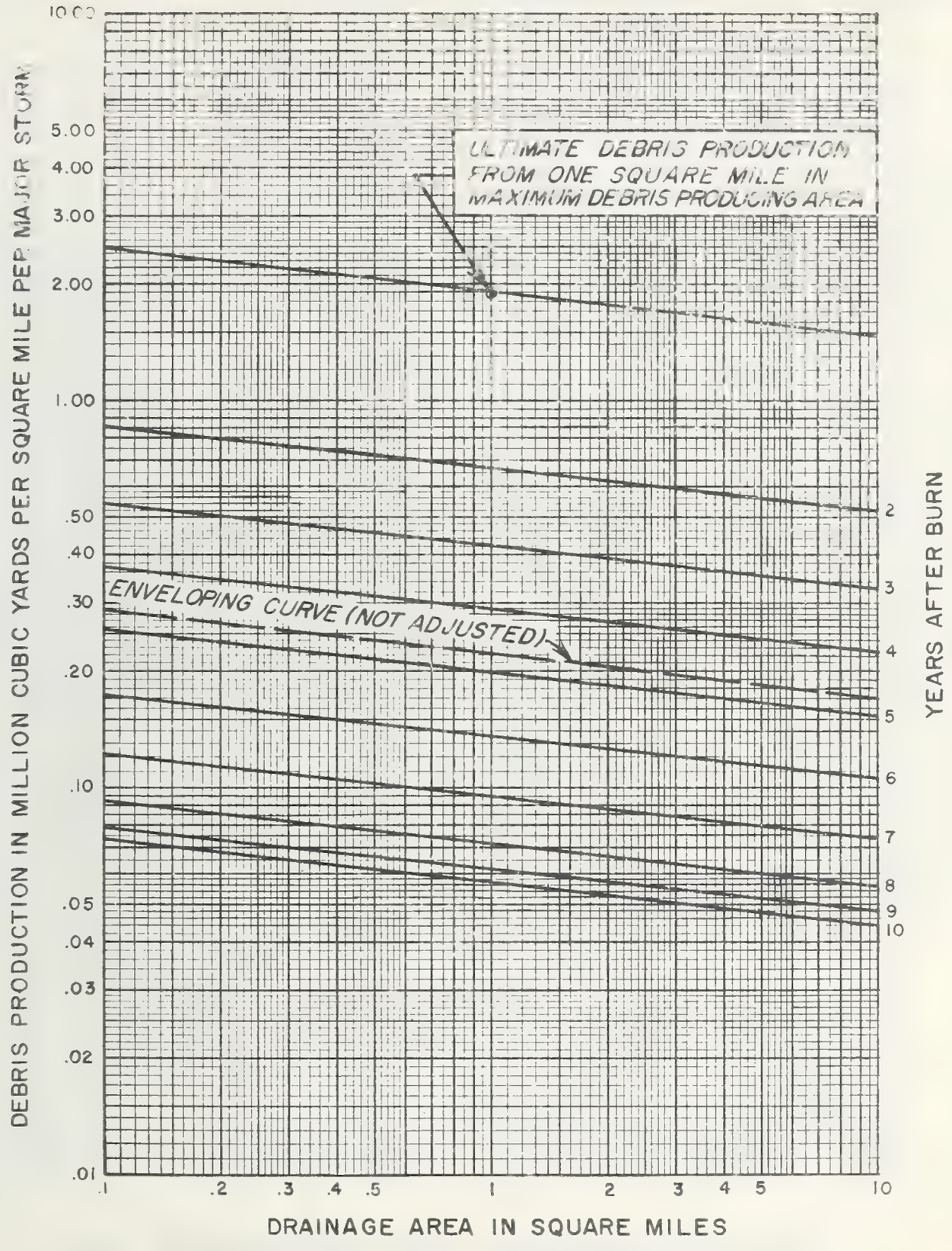


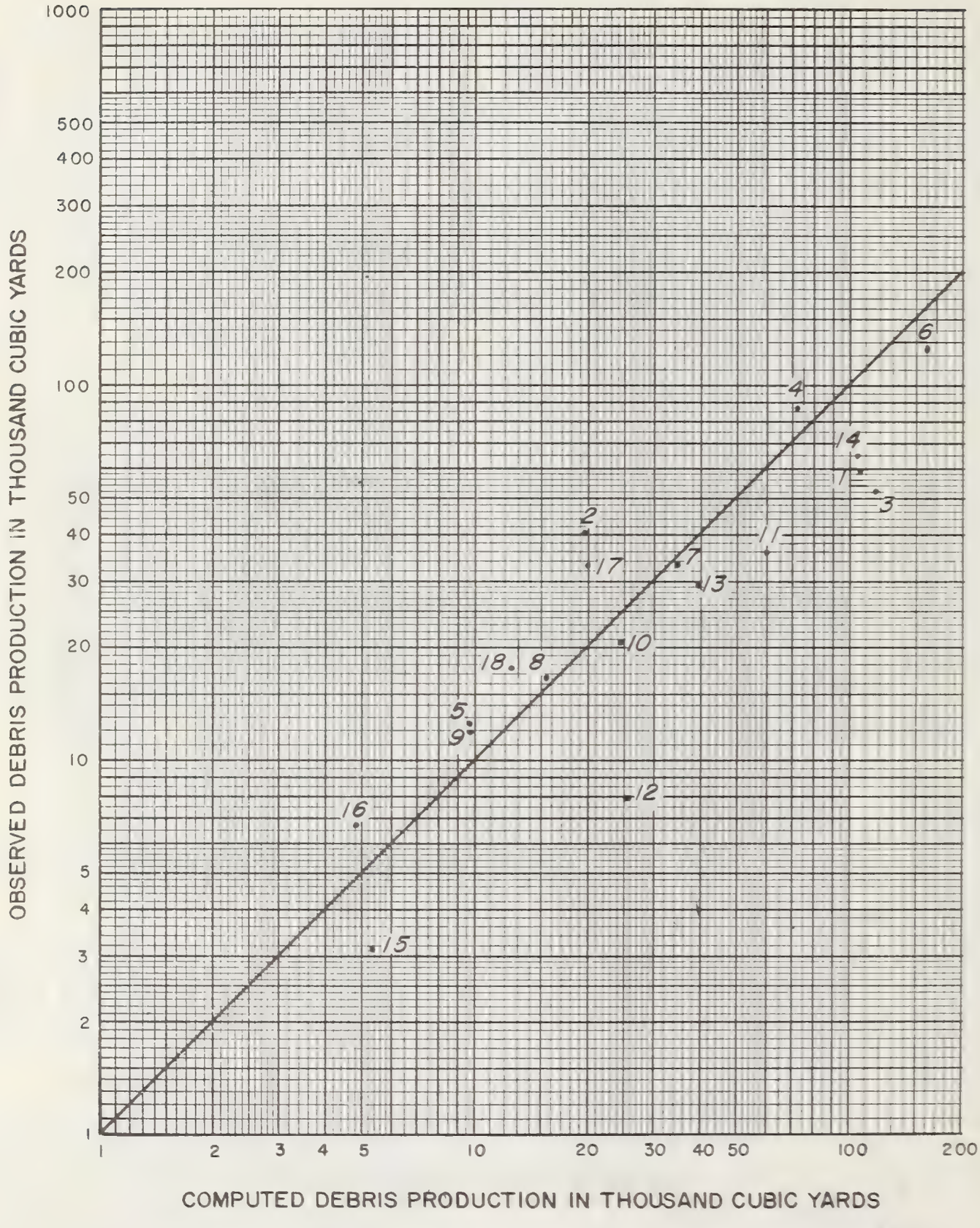
FIGURE 7. — Maximum 3-hour precipitation and hypsometric-analysis index curves indicating correction of debris production.



NOTE:

CURVES (BASED ON DATA FROM FIGURES 4 & 5) REPRESENT ULTIMATE DEBRIS PRODUCTION FOR INDICATED NUMBER OF YEARS AFTER 100 PERCENT BURN WITH ALL CORRECTION FACTORS ASSUMED AS 100 PERCENT.

FIGURE 8. — Ultimate debris production from maximum debris-producing area for one major storm.



NOTE:
NUMBERS INDICATE DEBRIS BASINS AS LISTED IN TABLE 2 AND DATA WERE TAKEN FROM COLUMNS 8 AND 21.

FIGURE 9. — Debris production relationship.

TABLE 3.—*Observed data and computed debris production from storm of 7 to 12, February 1962, Los Angeles area*

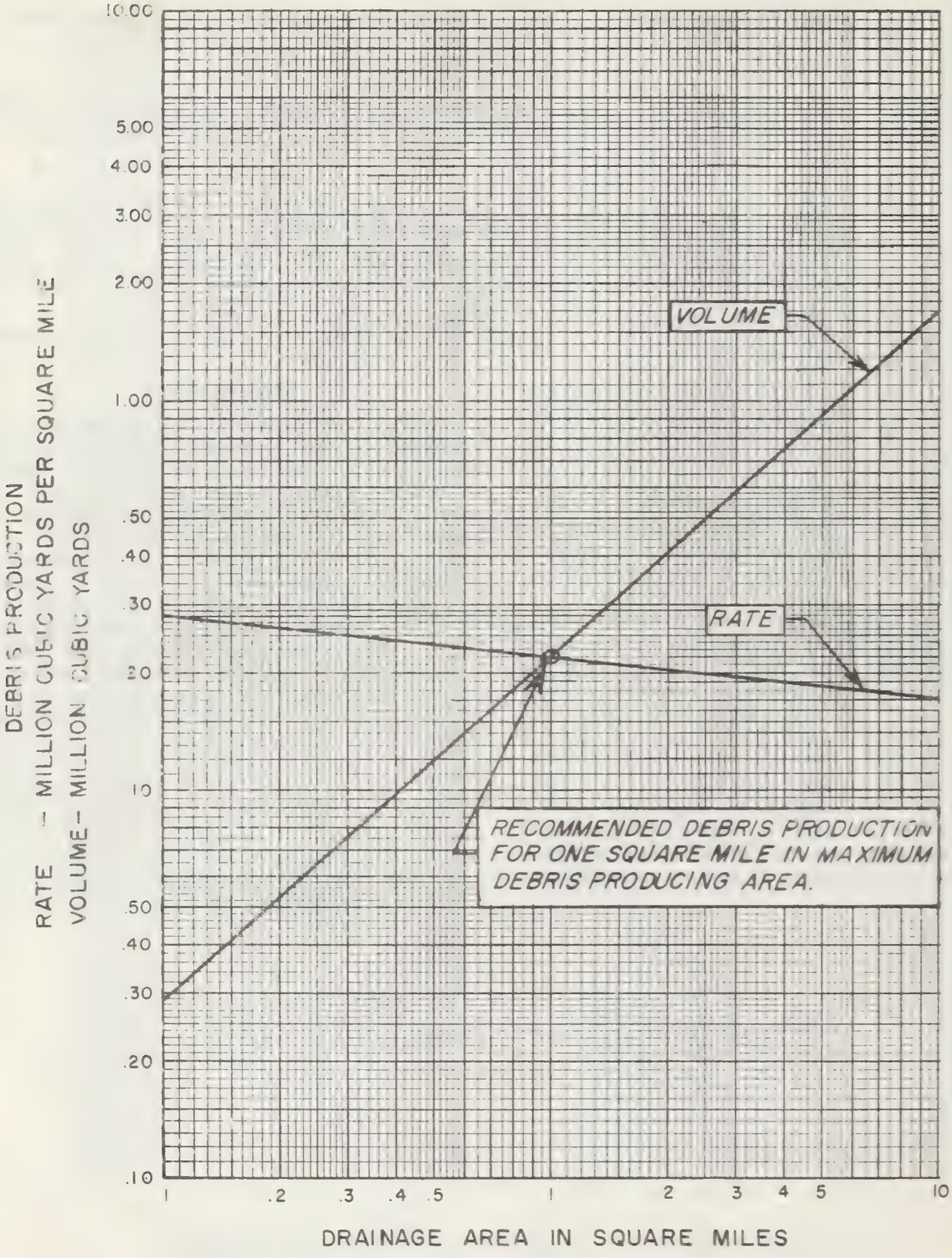
Debris basin[1]	Drainage area	Area burned	Observed debris production	Debris production factors				Correction factors					Computed debris production[2]
				Slope	Drainage density	Hypsometric index	3-hour rainfall	Slope	Drainage density	Hypsometric index	3-hour rainfall	Total	
	Square miles	*Percent*	*Cubic yards*	*Feet per mile*	*Miles per square mile*		*Inches*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Cubic yards*
Sierra Madre Villa	1.6	36	[3]	1,160	2.2	0.55	1.10	95	96	97	5	4.4	48,000
Bailey Canyon	.58	26	10,000	1,525	3.5	.62	1.10	100	80	85	4	2.7	9,300
Auburn Canyon	.21	80	20,100	2,240	0	.52	1.20	100	100	99	6	6.0	22,200
Carter Canyon	.11	100	11,700	2,540	3.5	.62	1.40	100	80	85	8	5.5	12,400

[1] See fig. 3 for location.
[2] Computed as explained in text under paragraph titled "Development of Adjustment Curves."
[3] Amount of debris produced not available. Total of 122,450 cubic yards of debris has accumulated since completion of the debris basin in February 1958.

ducive to runoff during the first year after 100 percent of the drainage area was burned, is believed to be too severe for design purposes. As indicated on figure 4, the debris potential of an area decreases rapidly in the first 5 years after a burn. The recommended design debris quantity from a maximum debris producing area of 1 square mile is 220,000 cubic yards, or about 12 percent of the maximum index value, unadjusted. This value is equivalent to the value for 1 square mile area obtained from the enveloping curve of debris inflow (fig. 1) increased by 100 percent. This value also represents the amount of debris that would be produced from a similar area if a major storm occurred, with ground conditions conducive to runoff, 4 to 5 years after 100 percent of the drainage area was burned (figs. 8 and 10). This value is then adjusted by the factors affecting the debris potential for the drainage area being studied and the resulting computed value is the capacity for design of the debris basin in question.

Conclusions

The method presented in this paper is based on using varying degrees of debris potential for the drainage area above each debris basin to determine such debris capacities for use in design of debris basins as will provide approximately equal protection to the different downstream areas. The method is believed to give reasonable values within the limits of presently available data. Obviously, where many interrelated variables are involved, a large number of basic data are required for definitive analysis. Further studies will be made as data become available. The method presented herein is believed applicable for other than the Los Angeles area; however, the correction factors should be verified. One of the basic assumptions in developing the method is that ground conditions, prior to the maximum 3-hour rainfall, are conducive to runoff. In order to use this method of computing debris potential for areas with various prior ground conditions, development of additional correction factors would be necessary.



NOTE:

CURVES SHOWN HEREON ARE BASED ON THE ASSUMPTION THAT THE DEBRIS PRODUCING STORM OCCURS 4 TO 5 YEARS AFTER 100 PERCENT BURN IN THE AREA.

FIGURE 10. — Recommended debris production from maximum debris-producing area for one major storm.

MEASUREMENT OF SEDIMENT DENSITY WITH GAMMA PROBES

[Paper No. 90]

By J. ROGER MCHENRY, *soil physicist,* and PAUL H. HAWKS, *geologist, USDA Sedimentation Laboratory, Agricultural Research Service, Oxford, Miss.*[1]

Introduction

Sedimentation studies generally require that the density of the sediment deposits be determined in order that estimates of deposition and accumulation can be made. The conversion of a measured volume of accumulated sediment to a weight basis is dependent on this information. Many types of soil and sediment samplers have been developed for taking undisturbed cores for volume-weight measurements. Numerous improvements have been made in the undisturbed core-type samplers, but their operation in underwater sediments has not been completely satisfactory (*4*).

Nuclear techniques have been employed in recent years for measuring the density and/or thickness of numerous materials (*5, 6*). The single-probe technique, which utilizes the reflection principle, has been successfully adapted to the measurement of soil density (*2, 9*), and of sediments in situ (*1, 3, 7*). This method has one disadvantage, which may, or may not, be serious, depending on the objectives of the particular study. The single-probe system measures the attenuation of the gamma rays, which, after emission from a radioactive source, are reflected or scattered back to the probe by the surrounding medium. The direct transmission of these gamma photons from the source to the detector tube must not occur, and is prevented by placing a lead shield between the source and the detector. In order to shield effectively the detector tube from the radioactive source, a shield of about 6 inches of lead is required. This physical separation of the source and detector in the vertical axis limits the density measurement to an ellipsoidal volume with a vertical axis of 12 to 18 inches. Thus, the determination of densities of narrow bands or near interfaces of sediments is not possible (*7*). In many cases this is not, however, a serious limitation of the method.

Van Bavel and coworkers (*10*) utilized the attenuation of gamma rays that passed through a medium between the source and detector to measure soil density. This procedure is known as the transmission method. The transmission method requires two access tubes or probes: one for the radioactive source, one for the detector. With proper instrumentation it is possible to measure the density of thin or narrow bands of material in situ. The transmission method of measuring density and thickness has been used extensively and successfully in industry (*5, 6*). The possibilities of using such a dual-probe system for the measurement of sediment densities in situ have been investigated and reported by this laboratory (*8*). This paper reports on the development of the dual-probe system and compares its performance in the laboratory and in the field with that of the single probe.

Methods and Materials

Gamma Probe

The single probe, henceforth called a gamma probe, used in this study was manufactured by Technical Operations, Inc.,[2] according to specifications furnished by the Beach Erosion Board (*1*). The original probe has been modified. This modification was largely one of streamlining the exterior container so that insertion into and removal from sediments was expedited. The gamma probe as used is shown in figure 1, together with necessary or desirable accessory equipment.

This particular probe employs a 3–millicurie radium–226 source located near the tip. The source is separated from the three Geiger-Mueller detector tubes by 6 inches of lead. The GM tubes are connected to a transistorized preamplifier, and from this signals are fed through a cable to a portable scaler, Nuclear-Chicago Model 2800.

Dual Probe

The dual-probe system, henceforth designated as the dual probe, consists of a gamma source, a scintillation detector, a fixed dual access-tube assembly for spacing the probe and source, and a suitable power supply and electronic read-out device. The equipment is shown in figure 2. The gamma radiation is produced by a 7–millicurie cesium–137 source placed in the smaller access tube. A NaI (Tl activated) scintillation crystal (1 inch in diameter, ½ inch high) is used to detect the gamma emissions. The crystal, photomultiplier tube, and transistorized preamplifier are housed in the probe in the larger access tube (2-inch diameter). The distance between the two access tubes is maintained constant, 12-inch center to center, by the jig arrangement shown in figure 2. The power supply and scaler-ratemeter used in this

[1] Research cooperative with the University of Mississippi and Mississippi State University

[2] Reference to commercial equipment or manufacturer is in no way an endorsement by the United States Department of Agriculture.

FIGURE 1. — A single gamma probe with operating accessories: (1) gamma probe containing the radioactive source, shielding, Geiger-Mueller detection tubes, and transistorized preamplifier; (2) aluminum connecting sections for inserting probe; (3) stopwatch; (4) personnel exposure pencils; (5) personnel exposure badge; (6) radiation meter; (7) portable scaler; (8) connecting cable; (9) carrying box, with lead shield, for gamma probe.

study was a Troxler Model 200B unit. This instrument is transistorized and when used in the field employs a silver-cadmium dry battery that can be recharged readily. In the laboratory this instrument can be operated from standard alternating current or from the battery. The Nuclear-Chicago scaler used with the gamma probe can also be operated in the laboratory from an alternating current supply.

Theory of Operation

The attenuation of a beam of gamma photons (rays) in matter follows the exponential relationship:

$$I = I_o\, e^{\frac{-\mu x}{\rho}} \qquad (1)$$

where I is the emergent intensity observed for an incident radiation intensity, I_o, in a material whose linear absorption coefficient is μ, its thickness x, and of density ρ. The absorption coefficient of a material is the product, $N\sigma$, of the number of atoms, N, per unit volume and of the cross section, σ. The mass absorption coefficient is $N\sigma/\rho$. The linear absorption coefficient, μ, is equal to $N\sigma$. In this paper μ is used as the total linear absorption coefficient. If μ and x are constant; that is, if the medium is homogeneous and geometry or distance is constant, the ratio I/I_o is inversely proportional to the density, ρ.

The exponential equation 1 is valid only when the emissions are monoenergetic and when the medium through which the emissions pass is homogeneous; that is, a constant μ. It is theoretically possible to obtain proportional values of $I_1, I_2, I_3, \ldots I_n$ for values of $\mu_1, \mu_2, \mu_3, \ldots \mu_n$ and sum them for total attenuation, but the procedure would be laborious. Practically, the medium to be tested should be homogeneous so that the necessary calibration information can be obtained readily. The requirement for monoenergetic emissions may be handled in either of two ways. Shielding of the source and detector may be employed so a minimum of scattered or reflected radiation reaches the detection system along with the unattenuated primary emission. In order to screen out effectively the undesired radiation, bulky and heavy shielding is re-

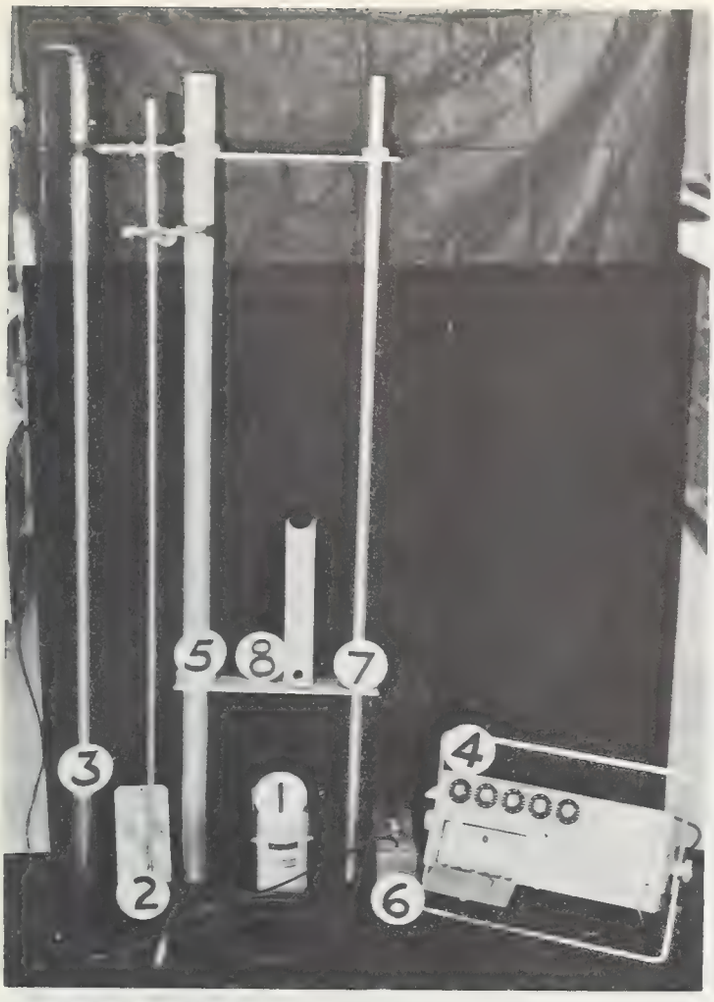


FIGURE 2. — A dual-probe assembly and accessories necessary for operation: (1) lead container for storage of radioactive source when not in use; (2) aluminum rod for inserting source in access tubing; (3) detector probe containing scintillation crystal, photomultiplier tube, and preamplifier and attached to a connector rod for insertion in access tube; (4) portable, transistorized, scaler-ratemeter; (5) aluminum access tube, 2-inch diameter, for detector probe; (6) battery charger; (7) aluminum access tube, ⅞-inch diameter for source; (8) standard magnesium bar for calibration, and resting on aluminum spacer for maintaining correct alinement of access tubes. Personnel exposure devices are not shown.

quired. This requirement presents problems in the field use of this method of controlling radiation detection. It is possible also to screen out unwanted radiation by the use of gamma scintillation spectrometry. All emissions below a certain energy level are discriminated against and are unrecorded when the scintillation spectrometer is operated integrally. Effectively, a "window" exists when the spectrometer is operated differentially. Electronic discrimination of energies below and above desired levels can be obtained with the differential spectrometer. To achieve this discrimination of observed radiation, complex electronic equipment is required. Units capable of performing these duties are available or are under development.

The gamma probe employs an empirical method that depends on the similar geometry of the test system and of the calibration setup, and on the existence of homogeneous medium about the probe that is similar to that of the calibration system. The direct emissions from the radioactive source are not detected by the detector tubes, but rather, the reflected or scattered radiations of lower energy are "seen." Under the conditions of the test; that is, identical geometry for test and calibration, the number of reflected photons reaching the detector tubes are inversely proportional to the density of the surrounding medium. The mass absorption coefficients of the elements of low atomic weights are similar. As a result, in systems of soil or sediment, and of water, the overall mass absorption coefficients of the constituents are so similar that the linear absorption coefficient may be substituted. The observed attenuation is then due to changes in density of the medium, as the average path of travel of the photons is unaffected by the small change in number of atoms per unit volume.

On strictly theoretical considerations, a calculation of the density of a medium based on the attenuation of reflected and scattered gamma photons is almost impossible. However, because of factors considered above, it is practical to determine the density of materials that approximate in makeup those for which calibration curves have been determined.

Measurements made by the dual probe can be based on theoretical considerations. Two things are required for this. First, the medium through which the photons will travel, and will be attenuated thereby, must be homogeneous. Second, the detection system must be capable of measuring the emitted monoenergetic gamma photons and only those monoenergetic gamma photons. This can be accomplished by combining a scintillation detector and a discrimination circuit in the electronic recording system. A gamma scintillation differential spectrometer would do just this. The Troxler 200B scaler as used was not capable of full discrimination. Partial discrimination was achieved, however, by adjusting the high voltage and amplifier controls. In effect, all pulses of less energy than the peak cesium–137 photons of 0.66 Mev. (million electron volts) were discriminated. This means that any scattered or reflected cesium–137 photons were not counted as they would be of reduced energy. Hence, the reduction in count rate between a standard and the test medium would be due to the differential attenuation of the emitted gamma photons.

Experimental Results

Laboratory

The procedure for calibration of the gamma probe for determining sediment densities has been described (*7*). A typical calibration curve is shown in figure 3. This particular curve was

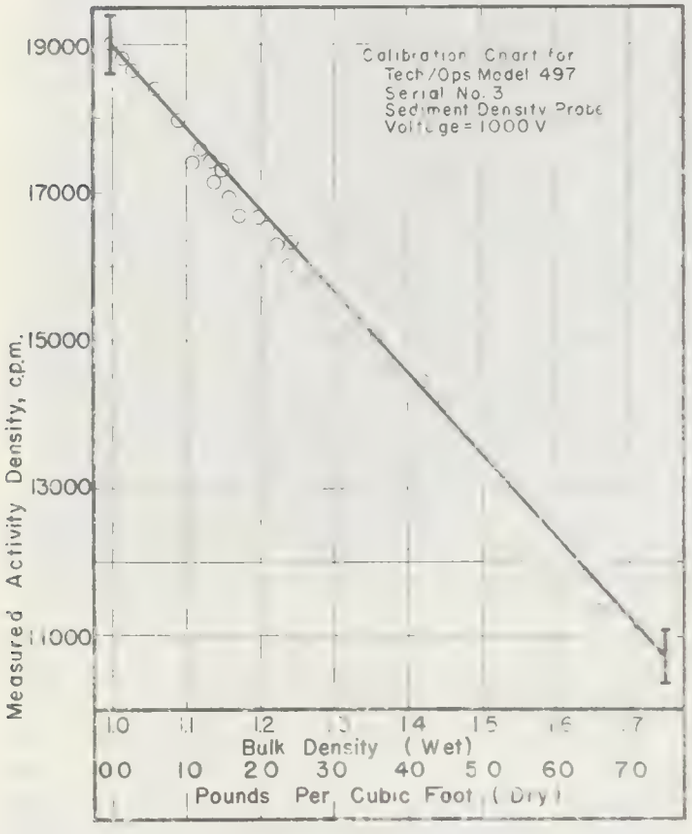


FIGURE 3. — A calibration curve for a single (gamma) probe. The measured activity density is plotted as a function of the density of the prepared sediment-water system. The conversion of density to pounds per cubic foot assumes a specific gravity of 2.65.

valid for only one gamma probe, Technical Operations, Inc., Serial No. 3, for a given period of time. Each gamma probe has a geometry peculiar to it, so each requires a calibration. The calibration curves obtained (*7*) are parallel, differing only in the magnitude of the measured activity density at a given sediment density. When a new Geiger-Mueller detection tube is installed in the probe or a transistor is replaced in the preamplifier circuit, a check on the applicability of the calibration curve is necessary. Usually, the curve will have shifted slightly from the previous level.

Note that in the calibration curve (fig. 3), the second, or lower, abscissa scale is applicable for sediments whose specific gravity approximates 2.65. Most sediments encountered in the Mississippi Valley have specific gravities approaching 2.65, that is, of silica or aluminosilicates.

The dual probe was calibrated according to directions of the manufacturer. A magnesium bar and water were used for this purpose. A reading was taken with the magnesium bar interposed between the source and the detection crystal. Then, a measurement was made with water occupying the intervening space. The density of the magnesium bar is 1.75. The measured activity density obtained with the magnesium bar was taken as 100 percent. The measured activity density of the water, specific gravity=1.00, was calculated as a percentage (350 percent) of the magnesium bar activity reading. A calibration curve was constructed from these two primary points. The resulting equation was:

$$\text{Sediment density} = 4.4945 - 1.3723x \quad (2)$$

(x = log of the measured activity density in counts per minute).

This calibration curve was applicable, as was that for the gamma probe, as long as the electronic components and geometry remained constant in the test system.

The utility of the calibration curve is shown in figure 4, where the measured sediment den-

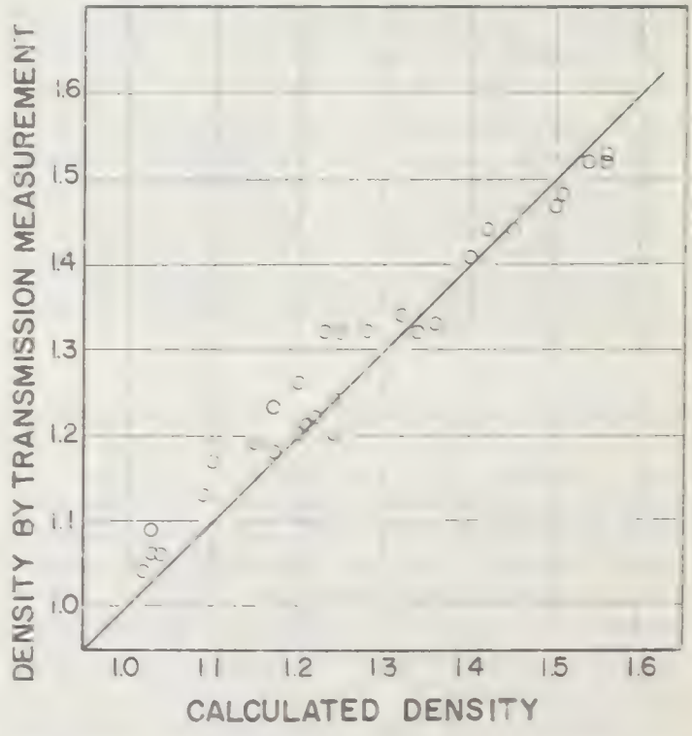


FIGURE 4. — The density of a prepared sediment measured by the dual-probe system plotted as a function of the calculated density. (Specified gravity assumed as 2.65.)

FIGURE 5. — Measuring the density of reservoir sediments. A dual probe, containing a radioactive source and a detection system, has been lowered from the raft into the sediment, and an activity density measurement is being made.

sity values are plotted against calculated sediment densities. Known weights of water and soil materials were mixed in containers, the volume measured, and the density computed. Then the dual probe was placed in the sediment and measurements of density made.

The fit of the observed points to the 1:1 ratio line is good. Many of the points that are somewhat off the line are values for those mixtures that contained considerable amounts of sand in systems of rather low viscosity. The amount of sand in the material employed as sediment was appreciable. The settling velocity of the coarse sand was sufficiently great in suspensions of low viscosity to affect the measurements of density. The data reported in figure 4 represent averages for 5-minute counting times. With careful control of the above-mentioned conditions, the observed densities very closely fit the 1:1 ratio line. To this end bentonite was added to increase the viscosity where possible and, if not, several shorter counting times were employed.

Field Studies

The gamma probe has been used in sedimentation surveys conducted by the Agricultural Research Service (*3, 7*). In Mississippi, a small reservoir near Oxford, called Power Line Reservoir, was surveyed in 1959 and 1960 and the gamma probe was used to measure sediment density. The Power Line Reservoir was used again in 1962 in a study to evaluate the utility of the dual probe for field surveys and to compare its performance with that of the gamma probe. All established ranges were resurveyed for sediment density, using both probes.

A specially constructed raft (fig. 5) was used in this survey. This raft is designed so that it may be quickly disassembled for transportation. Its utility for this purpose has been proved by use on a number of reservoirs of varying size and on the Gulf of Mexico.

The raft provides a stable platform for work; an overhead A-frame for the support of the probes and for the raising and lowering of the probes by means of the attached reel and cable; suitable facilities for using the nuclear equipment; and space for storing equipment when not in use. The center-well arrangement permits maximum accessibility to the probes by the crew. The umbrella provides shade for the electronic equipment, which is desirable both as protection against excessive heat and as an aid in reading the dekatron glow tubes of the scaler. On small reservoirs a cable is stretched across the designated range and the raft is attached to this cable and moved to the desired sample location. The distance moved is recorded on the productimeter attached to the raft and through which the cable passes. On larger bodies of water locating the raft is usually done by triangulation either from the raft or from a shore-based member of the field party.

The techniques employed in making density measurements with the gamma probe have been described (*3*, *7*). The techniques used with the dual probe were similar. The measurements of radiation activity were 1 minute in length. The absolute counting rate ranged from 6,000 to 30,000 counts per minute, yielding a standard deviation (σ) of 1.3 to 0.6 percent.

A comparison of typical data obtained by the two probes is shown in figure 6. Here the measured wet density is plotted as a function of sediment depth. As stated previously, the vertical height of the sediment layer measured, or "seen," by the dual probe is less than 1 inch. Likewise, the vertical resolution of the gamma probe is about 18 inches (*7*). The diameters of the circles used to indicate the dual-probe test points are approximately scaled to be the vertical dimension measured by the dual probe.

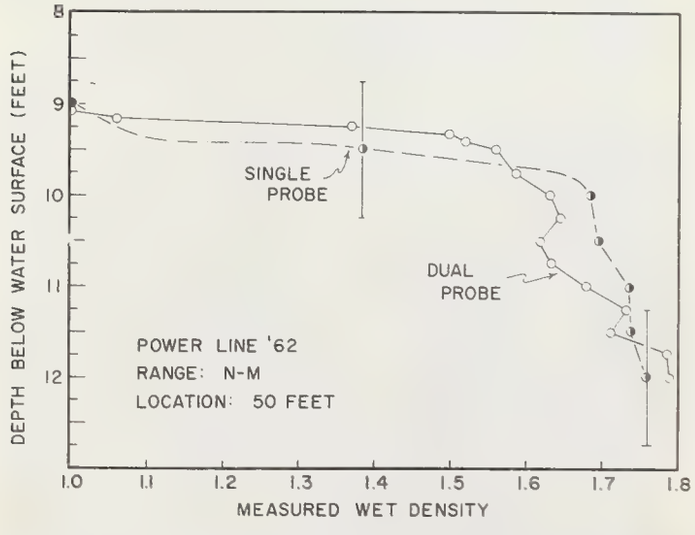


FIGURE 6. — A depth-density profile for a sediment as measured by the gamma and dual probes: Range N-M, 50 feet, Power Line Reservoir, Lafayette County, Miss.

The values for the density of sediments in situ obtained with the two probes are in general agreement. Allowances must be made for the integrating effect of the gamma probe as opposed to the specific density measure provided by the dual probe.

There is some indication of a semisolid sediment, or highly viscous suspension, overlying the consolidated sediment. In figure 6 the dual probe indicates that water exists at a depth of 9 feet 1 inch with sediments of densities of 1.059, 1.371, and 1.501 at 1-, 2-, and 3-inch lower depths. Considerable variation in the density of sediment with an increasing sediment depth was also noted. Typical depth-density profiles are shown in figures 7 to 10. In

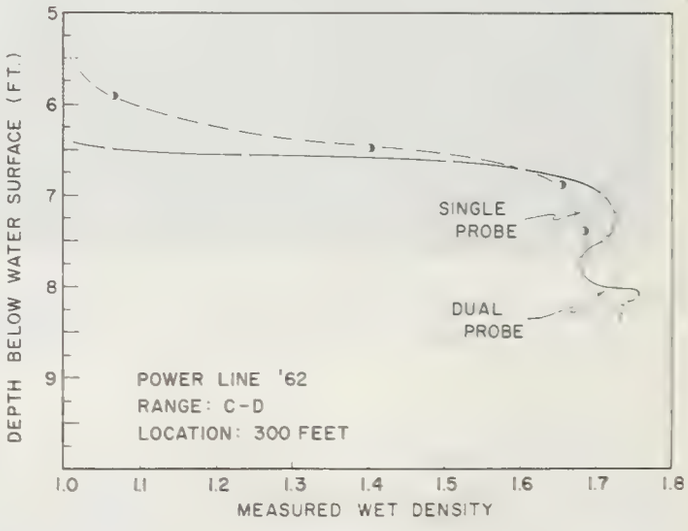


FIGURE 7. — A depth-density profile of a sediment as measured by a gamma and a dual probe: Range C-D, 300 feet, Power Line Reservoir, Lafayette County, Miss.

three of the four profiles the dual-probe density values are shown at greater depths than occur for the gamma probe. It is necessary to recall that the effective length of the gamma probe is 18 inches so that the bottom of the gamma probe is 9 inches below the illustrated, midpoint value of the gamma probe depth. In the case of the dual probe, the two access tubes are lowered as far as possible into the sediments. The lowest recorded dual probe value represents that of the source and detection tube at the bottom of the access tube.

In figure 7 the greater sediment densities at

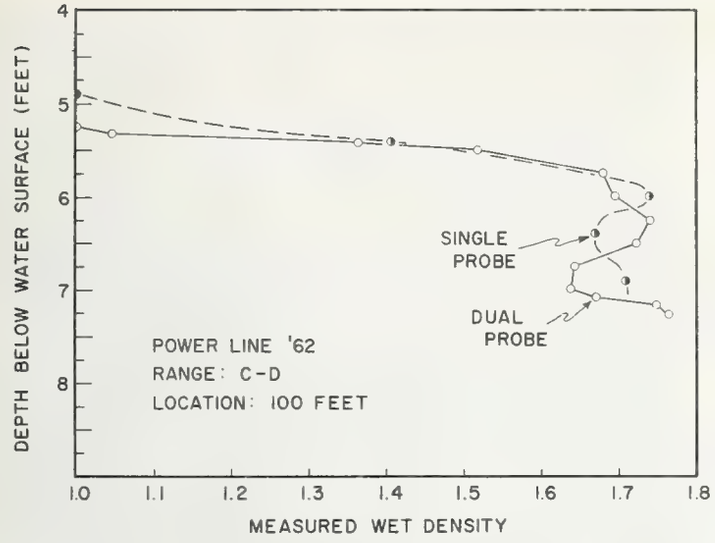


FIGURE 8.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range C-D, 100 feet, Power Line Reservoir, Lafayette County, Miss.

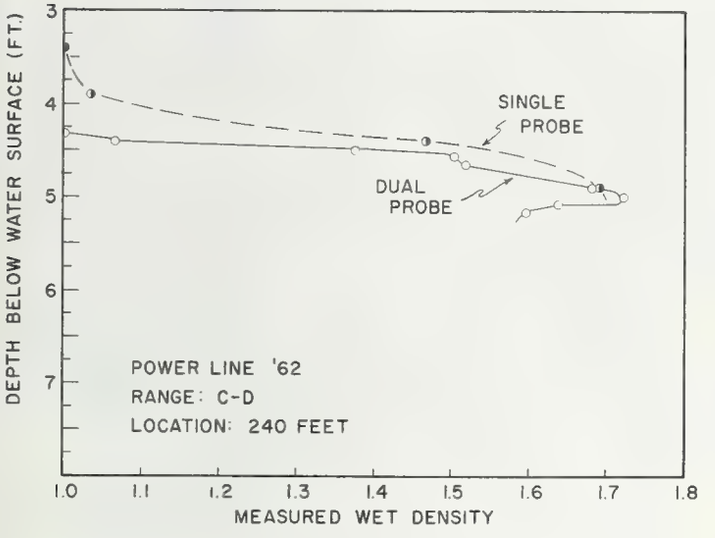


FIGURE 9.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range C-D, 240 feet, Power Line Reservoir, Lafayette County, Miss.

7 and 8 feet are not observed with the gamma probe. The lowest observed gamma probe value, 7 feet 5 inches, represents an integration of the sediment densities from 6 feet 8 inches to 8 feet 2 inches. One may arbitrarily assume that 60 percent of the observed attenuation of radiation occurs with ±3 inches of the center of the gamma probe; 30 percent with the second 3-inch layer; and that 10 percent occurs in the third 3-inch section. If these assumptions are applied to the data in figures 7 to 10, the results summarized in table 1 are obtained.

The agreement between the calculated and the observed integrated density values is reasonably good with the exception of the data taken from the 240-ft. profile in range C–D (fig. 9). In the latter there are insufficient dual-probe values to estimate adequately the density in an 18-inch vertical zone.

TABLE 1.—*A comparison of selected sediment densities obtained with a gamma probe with those calculated from dual-probe data*

Sample				Wet density	
Figure	Range	Location	Depth	Measured with gamma probe	Calculated from dual-probe data
		Feet	*Ft. In.*		
7 . . .	C–D	300	7 5	1.685	1.700
8 . . .	C–D	100	6 5	1.670	1.707
8 . . .	C–D	100	6 11	1.708	1.685
9 . . .	C–D	240	4 11	1.690	1.741
10 . . .	H–G	190	9 11	1.574	1.529

In three depth-density profiles (figs. 7, 8, and 9), the dual-probe data indicated variations in the density of the deposited sediments. In figure 8, the gamma probe also indicates this fact, although the influence of the integrating characteristic of the gamma probe reverses the true location of high and low density layers. Similar agreement in the measured density values by the two probes continued for other locations in range C–D in addition to those illustrated. At times, however, agreement was less good. In figure 10 considerable discrepancy is noted at the lower depths. This discrepancy and those in other data not shown may be due to any one or more of the following reasons. The location of the test spot is subject to variation. Lateral drift of the raft may occur due to wind currents. Even on quiet days, some

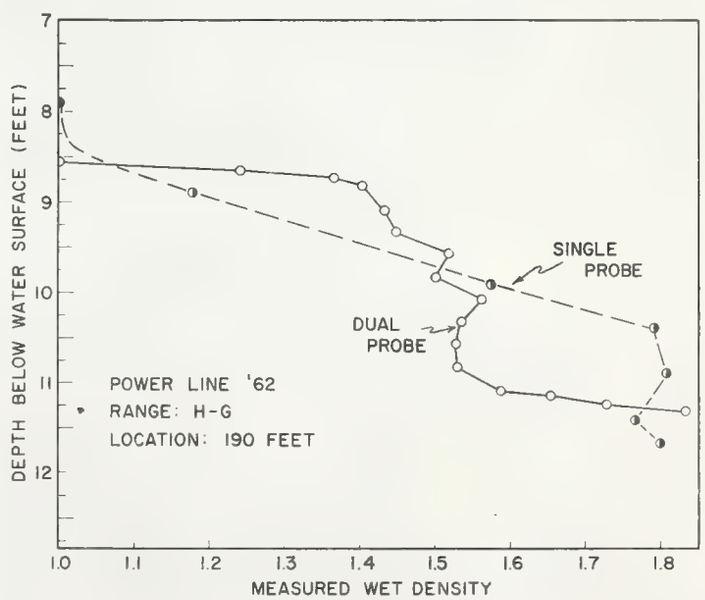


FIGURE 10.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range H-G, 190 feet, Power Line Reservoir, Lafayette County, Miss.

yaw of the raft occurs. The measurement of distance traveled on the cable is subject to error because slippage between the cable and productimeter may occur. Corrections for changes in the water level elevations were made. It is difficult, however, to measure the water level at the probe within an inch or two. The surveys with the two probes were made on different days, so relocation was subject to error. Although care was taken to insure that the probes were inserted vertically into the sediment, this was not always accomplished because of movements of the raft in the water. The data reported are assumed for duplicate determinations, but the actual points measured may be somewhat separated.

To summarize the representative data obtained with the dual probe, the sediment densities measured for the entire cross section C–D are shown in figure 11. The existence of a few inches of semiconsolidated sediment is seen, particularly between 140 and 180 feet. The high density sediments, those over 70 pounds dry weight per cubic foot, are confined to the edges of the larger channel and at the bottoms of the two smaller channels. Some penetration of the dual probe into the original ground is observed in the deep part of the illustrated cross section. Otherwise, little or no penetration of this material was achieved with this dual probe.

The existence of layers of sediment of different densities is readily apparent on examination of the data shown in figure 11. A number of factors have contributed to this layering, but discussion of them is beyond the scope of this paper. It was observed, however, in the course of the survey that those areas where densities were higher usually contained larger amounts of sand. The lack of high density material in the deeper parts of the cross section could, therefore, be ascribed to the settling out of the sand before the sediments were carried into the deeper pool areas.

Discussion

The performance of the dual probe has been generally satisfactory. In the laboratory, density measurements were made with the same precision and reliability as with the gamma probe. In the field, the performance of the dual probe was less dependable than was that of the gamma probe. This is due in part to the electronic circuitry comprising the scintillation detection system. Geiger-Mueller detector tubes, as employed in the gamma probe, function similarly over a range of applied voltages. Once this voltage plateau is reached, small changes in the applied voltage have little effect on the performance or sensitivity of the system.

On the other hand, the sensitivity of a scintillation system to radiation is directly proportional to the applied voltage — the greater the voltage, the greater the sensitivity and the more counts that are observed. In order, therefore, for a scintillation system to operate reliably, it is necessary to have rigid control of the high voltage supply. As presently constructed, the Troxler Model 200B does not provide adequate high voltage control for routine measurements. In the laboratory, and in the field when used by well-trained persons, the dual probe is presently adequate. For routine field work of the type where the gamma probe may now be used, the dual probe is inadequate in its present state. However, within the noted limitations, its performance is adequate and its use permits the collection of unique data. As auxiliary portable equipment becomes available, the performance of the dual probe will be improved.

Summary and Conclusions

The use of nuclear techniques for measuring the densities of materials by the attenuation of gamma rays has been successfully applied to reservoir sedimentation studies. The performance of a single (gamma) probe and of a dual-probe (gamma) system for measuring densities of reservoir sediments in situ was evaluated. Both field and laboratory tests were conducted.

Previous work has indicated the gamma probe is unsuitable for measuring the densities of sediments in narrow layers or near phase interfaces. The dual probe provides means whereby such measurements can be made. The reliability and precision of these measurements is equal to that obtained with the gamma probe for larger masses of sediments.

In its present state of development, the dual probe is capable of measuring the density of 1-inch layers of sediment unaffected by the material above or below. Such measurements of density have been made successfully in up to 3 feet of sediment beneath 10 to 15 feet of water.

The performance of the dual probe in the laboratory was entirely satisfactory. However, the field performance of the dual probe in its present state of development is not equal to that of the gamma probe. An increase in the ruggedness of the design and in the reliability of the observed measurements is necessary.

The performance of the gamma probe is such that its use in routine sediment surveys is now possible. Additional design modifications and improved measurement techniques must be devised before the dual probe can be employed in the average sedimentation survey. However, for special purpose surveys, the dual probe can now be employed by suitably trained personnel.

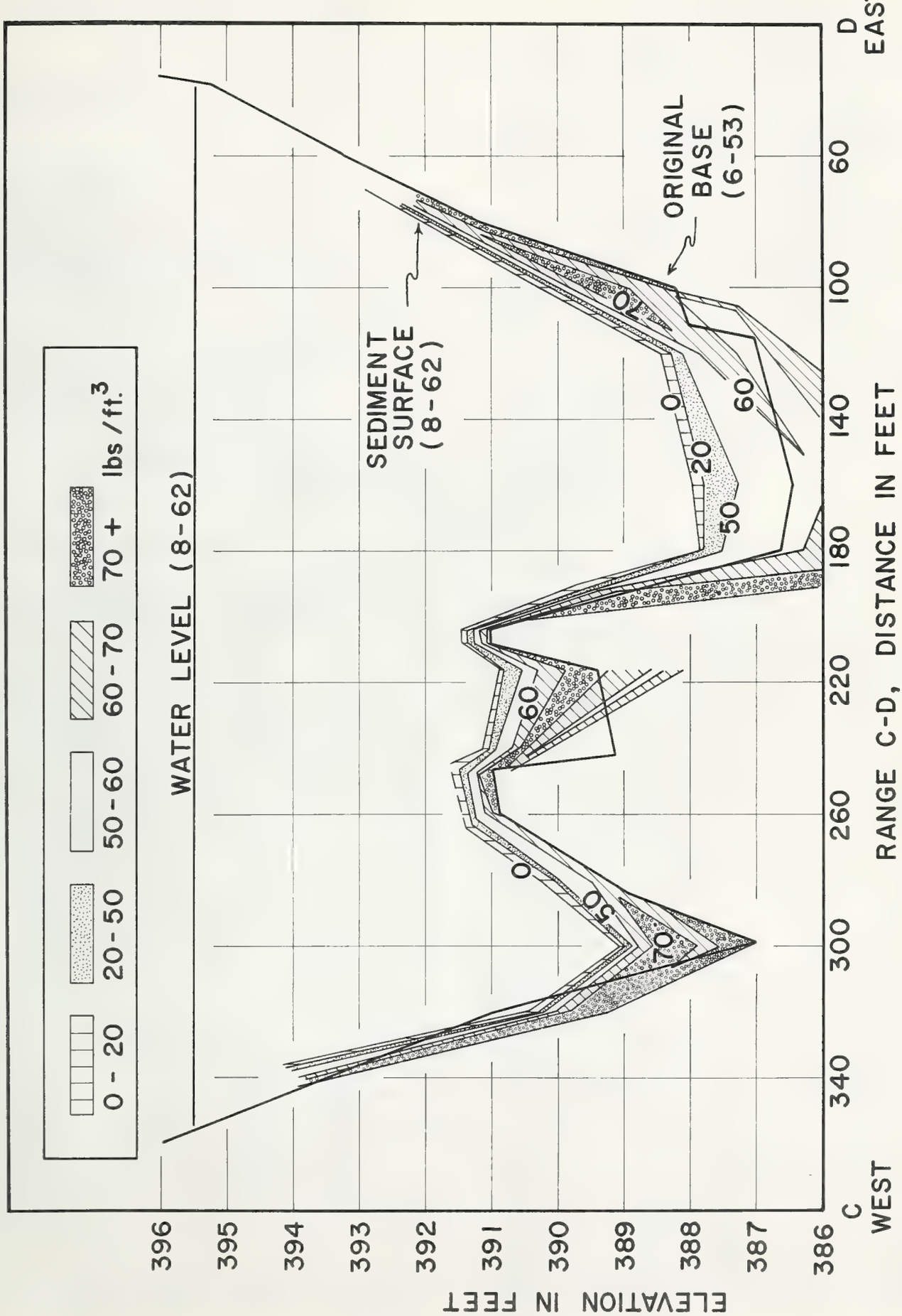


FIGURE 11.—Sediment density, in pounds per cubic foot of dry material, measured by the dual probe: Range C-D, Power Line Reservoir, Lafayette County, Miss., 1962.

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SEDIMENT WITHDRAWAL INVESTIGATION — GUERNSEY RESERVOIR

[Paper No. 91]

By E. A. JARECKI, *hydraulic engineer, Region 7,* and T. D. MURPHY, *hydraulic engineer, Office of Chief Engineer, Bureau of Reclamation, Denver*

Synopsis

Guernsey Reservoir, which was constructed primarily for irrigation storage, was drawn down purposely to produce increased sediment concentration releases. Data were collected for 3 years to determine sediment movement and redistribution of substantial sediment deposits within the reservoir area. Inflow and outflow rates, as well as the drawdown rate, were varied to establish sediment outflow from the reservoir under various conditions. Studies of field data were made to estimate the efficiency and length of life that can be expected from future reservoir drawdowns with respect to removing sediment from the reservoir.

Results indicate that controlled fluctuated reservoir inflow (clear water) does not, in itself, significantly improve the maximum sediment concentration outflow. The relative efficiency of sediment withdrawal by reservoir drawdown has varied considerably with fluctuation of reservoir storage and redistribution of sediment within the reservoir. Sediment withdrawal is greater at low reservoir storage and will be limited if a storage cushion is required at the bottom of a reservoir for power operations. Increased concentrations of sediment withdrawn from the reservoir can be obtained by increases in inflow at minimum storage. It is expected that a stable channel gradient will ultimately prevail through the reservoir and, combined with some continued channel widening, will provide sufficient sediments for withdrawal if reservoir storage can be decreased below 2,000 acre-feet and inflows can be fluctuated.

Introduction

For many years, the irrigation systems in the North Platte area have experienced heavy intake of sediments with the diverted canal waters. Canal cleaning problems were severe, but canal seepage losses appeared to be less. With the construction of Guernsey Dam and Reservoir in 1927, clear water releases tended to increase the canal seepage losses and canal bank erosion. As sediment accumulation increased in Guernsey Reservoir, normal reservoir operations during the past decade removed sediments from the reservoir and again beneficial aspects appeared in the downstream canals and laterals. With the construction of Glendo Dam and Reservoir in 1957, about 16 miles upstream, sediment inflow to Guernsey Reservoir was radically reduced. With an integrated operation of the North Platte River storages for irrigation and power purposes, it was necessary to evaluate the qualitative and quantitative extent of sediment withdrawals from Guernsey Reservoir, the effectiveness of the sediment-laden water upon canal seepage, and the most effective means of extracting the reservoir sediments — now and in the future.

General Description

Guernsey Dam, on the North Platte River in Wyoming, was completed by the Bureau of

Reclamation in 1927, as a feature of the North Platte Project. The reservoir behind the dam is used for the storage of irrigation water, reregulating upstream storage releases, and for power production. Irrigation releases from Guernsey Reservoir to downstream lands average about 1,130,000 acre-feet per year. The present surface area of the reservoir is about 2,380 acres at crest elevation of 4,420; the existing storage at this elevation is about 44,800 acre-feet, as compared to the original capacity of 73,810 acre-feet. The length of the reservoir is 14.6 miles; the width varies from less than 500 feet to approximately ¼ mile for a distance of about 5 miles above the dam. Above this reach, the reservoir for over a 4-mile reach widens, ranging from ½ mile to almost 1 mile. The upper reach of the reservoir is narrow, ranging from 300 feet to about ¼ mile.

Although the North Platte drainage area above the Guernsey Dam is about 16,200 square miles, the sediment contributing area has varied from 5,500 square miles in 1927 (below Pathfinder Dam), 5,400 square miles in 1938 (below Alcova Dam) and to about 700 square miles in 1957 (below Glendo Dam). Up to 1957, the reservoir has been subject to a very high sedimentation rate, losing 39.3 percent of the original capacity. Since construction of Glendo Dam, or since 1957, the estimated rate

TABLE 1.—*Sediment concentration data for reservoir drawdown periods, Guernsey Reservoir 1959–62*

Date	Above Guernsey		In Guernsey			Below Guernsey
	Sta I	Sta II	Sta IV	Sta V	Sta VI	Sta III
1959	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
Aug. 6						114
Aug. 7		90		4,270	3,750	476
Aug. 8	18	68				847
Aug. 9						585
Sept. 16	8	95				10
1960						
July 2,3,4	11	11				11
July 7						11
July 9	14	14				18
July 10	12	11	600	20	6	33
July 11		14	1,060	150	25	33
July 12	11	14	560	730	50	84
July 13		15	560	5,850	530	310
July 14		17	375	4,930	950	235
July 15	11	28	360	2,620	660	310
July 16	11	14	355	2,900	2,050	169
July 17		11	330	2,140	4,160	202
July 18		11	332	2,140	5,970	264
July 19		11	352	2,140	3,000	255
July 20						156
July 21						88
July 22						90
July 23						69
July 24						62
July 25						50
July 26						28
July 27						11
July 28						17
July 29						15
Aug. 4	22	23				18
Aug. 5						15
Aug. 6						25
Aug. 7	19	15				19
Aug. 8		18	370	25	17	21
Aug. 9		18	350	70	20	61
Aug. 10		22	550	160	35	63
Aug. 11		29	600	240	55	88
Aug. 12	28	29	620	530	340	123
Aug. 13		16	470	2,600	570	178
Aug. 14		23	525	2,190	3,080	369
Aug. 15		26	560	1,830	1,180	195
Aug. 16		18	352	1,060	1,260	120
Aug. 17		23	598	575	1,330	154
Aug. 18		31	580	1,700	1,575	134
Aug. 19	30					95
Aug. 20						57
Aug. 21						65
Aug. 22						71
Aug. 23	21					76
Aug. 24						82

Date	Above Guernsey		In Guernsey			Below Guernsey
	Sta I	Sta II	Sta IV	Sta V	Sta VI	Sta III
1960	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
Aug. 31	19					
Sept. 9	15					
1961						
July 8						15
July 18						14
July 20		12	300	54	48	
July 22						39
July 23		7	400	190	54	40
July 24		10	180	300	56	34
July 25				300	74	46
July 26		13	161	3,900	255	86
July 27	46	74	840	1,250	4,600	390
July 28		12	225	1,100	2,400	460
July 29			330	1,370	2,930	540
July 30			175	900	1,500	560
July 31			260	770	335	76
Aug. 1				900	52	37
Aug. 2				125	56	34
Aug. 3			105	74	40	24
Aug. 7		7	350	29	50	10
Aug. 20						13
1962						
July 12	21					
July 21						27
July 22						50
July 23						52
July 24			61	440	235	
July 25						44
July 26		26	39	84	370	40
July 27			100	210	285	34
July 28		20	130	720	270	60
July 29			92	2,900	460	125
July 30			88	1,700	2,000	200
July 31			66	1,060	880	190
Aug. 1			87	1,140	3,750	410
Aug. 2			115	920	2,050	780
Aug. 3			103	920	1,900	780
Aug. 4		20	102	810	1,375	590
Aug. 5			145	870	1,405	620
Aug. 6			105	990	1,610	710
Aug. 7		25	90	890	1,410	690
Aug. 8			160	1,270	1,720	770
Aug. 9			94	310	270	115
Aug. 10			70	79	365	65
Aug. 11			35	31	52	32
Aug. 12		28	29	81	56	22
Aug. 13						15
Aug. 21						18
Aug. 22						15
Aug. 31						14

of sediment inflow is approximately 150 acre-feet per year as compared to the historic sediment accumulation of 970 acre-feet per year.

Available Data

Under the Bureau of Reclamation's Soil and Moisture Conservation Program, data pertaining to sediment concentration of reservoir inflow, sediment movement within the reservoir proper, and sediment concentration of reservoir outflow were collected relative to the Guernsey Reservoir drawdown periods 1960, 1961, and 1962. These data are summarized in table 1. Data relative to the reservoir drawdown periods are tabulated in table 2. A general graph of reservoir operations from 1957 to 1962 and pertinent sediment concentration data collected during the withdrawal periods

TABLE 2.—*Guernsey Reservoir drawdown data, 1959–62*

Date (1)	Water surface elev. (2)	Storage (3)	Inflow (4)	Outflow (5)	Difference, column 4−5 (6)	Maximum concentration inflow (7)	Concentration outflow (8)	Trap efficiency, (8)÷(7) (9)
1959	*Feet*	*Acre-feet*	*C.f.s.*	*C.f.s.*	*C.f.s.*	*P.p.m.*	*P.p.m.*	*Percent*
Aug. 7, 1959	4,376.6	2,600	4,948	5,190	−242	4,270	476	11.1
1960								
July 10	4,405.6	16,900	2,916	4,960	−2,044	600	33	5.5
July 11	4,399.2	10,300	2,886	5,000	−2,114	1,060	33	3.1
July 12	4,392.2	5,500	2,868	5,000	−2,132	730	84	11.5
July 13	4,386.6	3,200	3,975	4,980	−1,005	5,850	310	5.3
July 14	4,383.0	2,200	4,424	4,980	−556	4,930	235	4.8
July 15	4,384.9	2,700	5,155	4,980	+175	2,620	310	11.8
July 16	4,383.4	2,300	4,800	5,020	−220	2,900	169	5.8
July 17	4,381.4	1,750	4,796	5,040	−244	4,160	202	4.8
July 18	4,380.9	1,650	4,987	5,040	−53	5,970	264	4.4
July 19	4,389.1	4,150	6,105	5,080	+1,025	3,000	255	8.5
Aug. 8	4,404.98	16,000	3,981	4,800	−819	370	21	5.7
Aug. 9	4,403.29	14,100	3,705	4,820	−1,115	350	61	17.4
Aug. 10	4,401.19	12,000	3,616	4,840	−1,224	550	63	11.4
Aug. 11	4,398.25	9,500	3,423	4,880	−1,475	600	88	14.7
Aug. 12	4,394.00	6,600	3,676	4,820	−1,144	620	123	19.8
Aug. 13	4,388.3	3,800	3,613	4,710	−1,097	2,600	178	6.8
Aug. 14	4,381.0	1,650	4,404	4,630	−226	3,080	369	12.0
Aug. 15	4,381.5	1,750	4,361	4,480	−119	1,830	195	10.6
Aug. 16	4,382.2	1,950	4,224	4,360	−136	1,260	120	9.5
Aug. 17	4,380.4	1,550	4,194	4,350	−156	1,330	154	11.6
Aug. 18	4,381.1	1,700	5,850	4,270	+1,580	1,700	134	7.9
1961								
July 23	4,400.7	11,600	3,979	4,570	−591	400	40	10.0
July 24	4,399.2	10,300	3,662	4,570	−908	300	34	11.3
July 25	4,396.35	8,050	3,010	4,590	−1,580	300	46	15.3
July 26	4,389.40	4,250	3,136	4,610	−1,474	3,900	86	2.2
July 27	4,380.10	1,500	4,258	4,590	−332	4,600	390	8.5
July 28	4,378.90	1,200	4,565	4,610	−45	2,400	460	19.2
July 29	4,378.00	1,050	3,902	3,960	−58	2,930	540	18.4
July 30	4,378.30	1,100	4,534	4,160	+374	1,500	560	37.3
July 31	4,384.00	2,450	2,933	2,340	+593	770	76	9.9
Aug. 1	4,388.40	3,900	2,347	1,750	+597	900	37	4.1
Aug. 2	4,392.90	5,900	2,446	1,280	+1,166	125	34	27.2
Aug. 3	4,398.30	9,500	3,797	1,030	+2,767	105	24	22.9
Aug. 7	4,407.1	19,000	4,058	4,230	−172	350	10	2.9
1962								
July 26	4,404.77	16,800	3,596	4,720	−1,124	370	40	10.8
July 27	4,402.95	14,600	2,846	4,631	−1,785	285	34	11.9
July 28	4,398.0	9,800	3,049	4,479	−1,430	720	60	8.3
July 29	4,390.8	4,800	3,230	4,422	−1,192	2,900	125	4.3
July 30	4,388.2	3,550	3,928	4,270	−342	2,000	200	10.0
July 31	4,385.6	2,500	3,208	3,791	−583	1,060	190	17.9
Aug. 1	4,380.1	1,500	3,348	3,740	−392	3,750	410	10.9
Aug. 2	4,377.9	1,150	3,442	3,536	−94	2,050	780	38.0
Aug. 3	4,377.1	1,000	3,555	3,519	+36	1,900	780	41.0
Aug. 4	4,377.9	1,150	3,759	3,723	+36	1,375	590	42.9
Aug. 5	4,378.0	1,150	4,241	4,090	+151	1,405	620	44.1
Aug. 6	4,379.6	1,400	4,858	4,840	+18	1,610	710	44.1
Aug. 7	4,379.2	1,350	4,758	4,726	+32	1,410	690	48.9
Aug. 8	4,380.7	1,600	6,089	4,490	+1,599	1,720	770	44.8
Aug. 9	4,395.4	7,800	7,176	4,669	+2,507	310	115	37.1
Aug. 10	4,401.8	13,300	7,007	4,783	+2,224	365	65	17.8
Aug. 11	4,406.9	19,800	6,772	4,840	+1,932	52	32	61.5
Aug. 12	4,407.7	20,700	5,246	4,745	+501	81	22	27.2

are shown in figure 1. Included in the investigations during the drawndown periods were information on bank erosion of the exposed sediment deposits, channel cross sections in the reservoir area, water surface slopes, and a resurvey of ranges in the lower end of the reservoir. The 1957 resurvey of Guernsey Reservoir was used in obtaining prior data on reservoir sediment ranges, size analyses, and densities of reservoir sediment deposits.[1]

Sediment Characteristics

Suspended sediment samples were obtained within the reservoir study reach and from the reservoir releases during the withdrawal investigations. The average size analyses curves for these respective locations are shown in figure 2. Sampling of the reservoir sediment deposits was also performed during these periods. Samples were taken of the channel bed material and the quasi overbank deposit within the reservoir area (fig. 2).

Unit weight determinations were based on samples of reservoir sediment deposits, gamma probe investigations of in situ sediment deposits, and computations of volume weights relative to the size analyses of suspended sediments. An average unit weight of 67 pounds per cubic foot was determined from these methods and was employed in the analysis of the sediment withdrawal data.[2]

Reservoir Sediment Distribution

Resurvey of the reservoir in 1957 determined a sediment accumulation of 29,100 acre-feet in the reservoir proper. Based on inflow-outflow sediment concentration data collected during the period 1957–1962, an estimated 117 acre-feet of sediment has been removed since the 1957 survey. Of the 117 acre-feet total sediment removed, 88 acre-feet were removed during the withdrawal periods 1960, 1961, and 1962, or approximately 75 percent of the total removed. The longitudinal distribution of sediment deposits determined at the various reservoir resurveys and investigations is shown in figure 3. The thickness of the sediment depth has ranged up to almost 40 feet. The thalweg profile depicts the slope of the deposits at various locations in the reservoir area and their change relative to time. The slope of the delta topset bed is estimated to be about 2.5 feet per mile as compared to 5.7 feet per mile for the original river channel slope.

Comparing the 1957 profile with subsequent profiles (fig. 3) shows a decrease in the upper reservoir sediment deposits and an increase in sediment deposition in the lower portion of the reservoir. Below elevation 4,386 feet, there was a reduction in reservoir capacity of about 31 percent. A typical cross section of sediment deposits in the lower reservoir reach is shown in figure 4. The redistribution of sediment deposits during the 5-year interim period occurred relative to drawdown periods, at which time the main channel traversing the reservoir area scoured the upper sediment beds and conveyed the bulk of the scoured sediment to the lower portion of the reservoir. This movement, however, applies in general to the thalweg, as defined by the single profile shown for respective years.

Sediment Withdrawals

The aforementioned data, tables 1 and 2, were analyzed to determine possible correlations, if any, between inflow-outflow rates, drawdown rates, or storage fluctuation on the sediment concentration of the reservoir discharge. The parameter used for determining the effect of the drawdown operation will be referred to as relative efficiency, being defined as the ratio of the observed suspended sediment concentration of the reservoir releases to the maximum observed suspended sediment concentration inflow to the reservoir. The maximum observed inflow concentrations to the reservoir occurred in proximity to the reservoir estuary, which fluctuated during and between drawdown periods.

Effect of Inflow-Outflow Rates

During the drawdown investigations, the reservoir inflow ranged between approximately 2,000 and 7,000 c.f.s. with controlled releases from the upstream Glendo Reservoir. The respective inflow-outflow discharges for the drawdown periods 1960-62 are tabulated in table 2 and are shown graphically in figures 5, 6, and 7. The efficiency (col. 9, table 2) appears to be affected by inflow-outflow fluctuations; however, there appears to be no significant trend. The relative efficiency of the sediment withdrawal is somewhat higher when the inflow exceeds the outflow; however, this does not give an entirely independent parameter. The effect of increased inflow, coupled with minimum reservoir storage, does appear to have impact on relative efficiency and absolute maximum sediment inflow and outflow. These effects are to be discussed later in this paper.

Effect of Drawdown Rate

The drawdown rate in acre-feet per day was determined from the respective slopes of the drawdown storage curves shown in figures 5, 6, and 7. The storage curves are based on revised

[1] BUREAU OF RECLAMATION. SEDIMENTATION RESURVEY OF GUERNSEY RESERVOIR, 1957, NORTH PLATTE PROJECT, WYOMING AND NEBRASKA. Sedimentation Section, Hydrology Branch. August 1958.

[2] See paper 82 of this Conference.

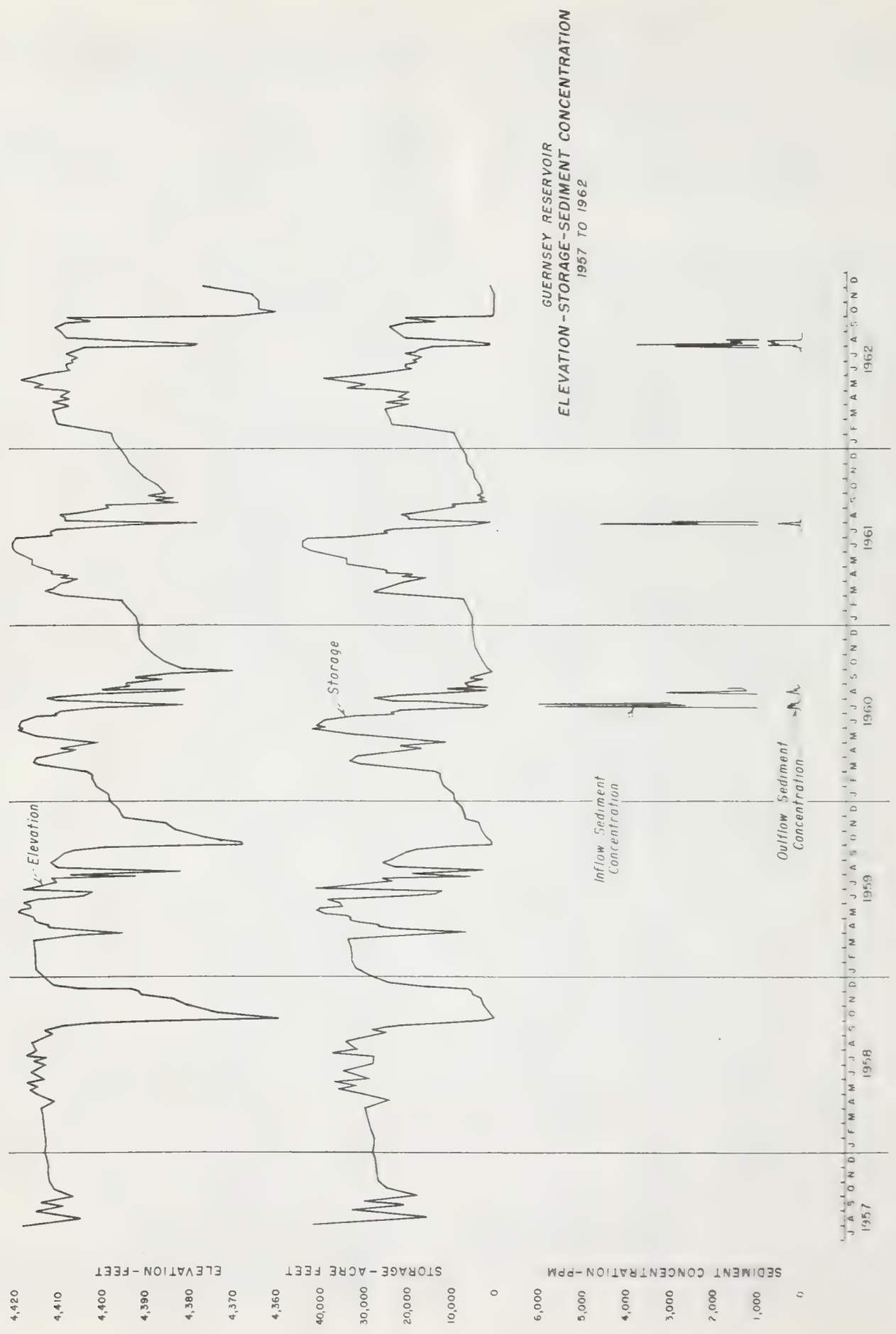

FIGURE 1. — Elevation-storage-sediment concentration, 1957 to 1962.

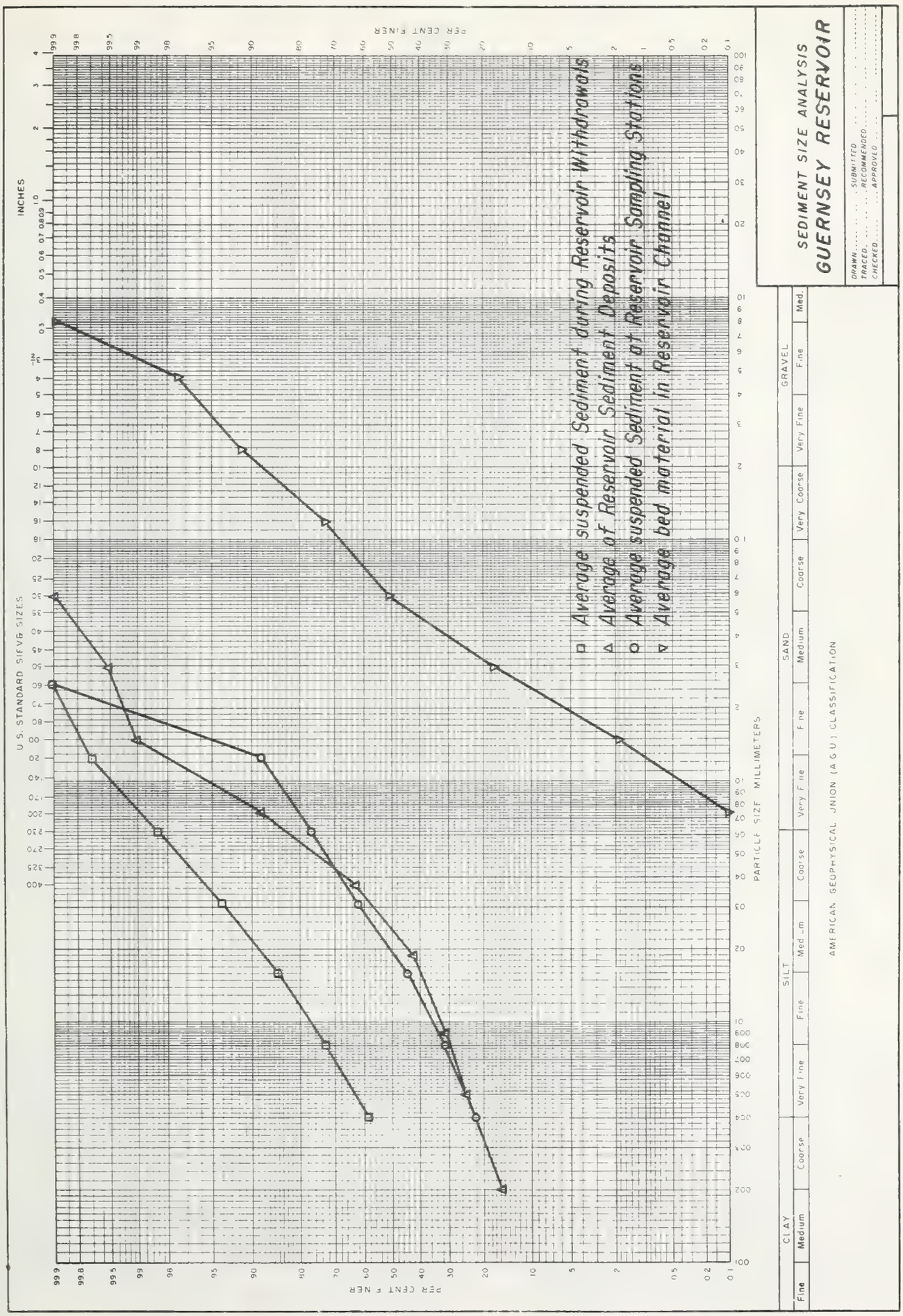

FIGURE 2. — Sediment size analysis.

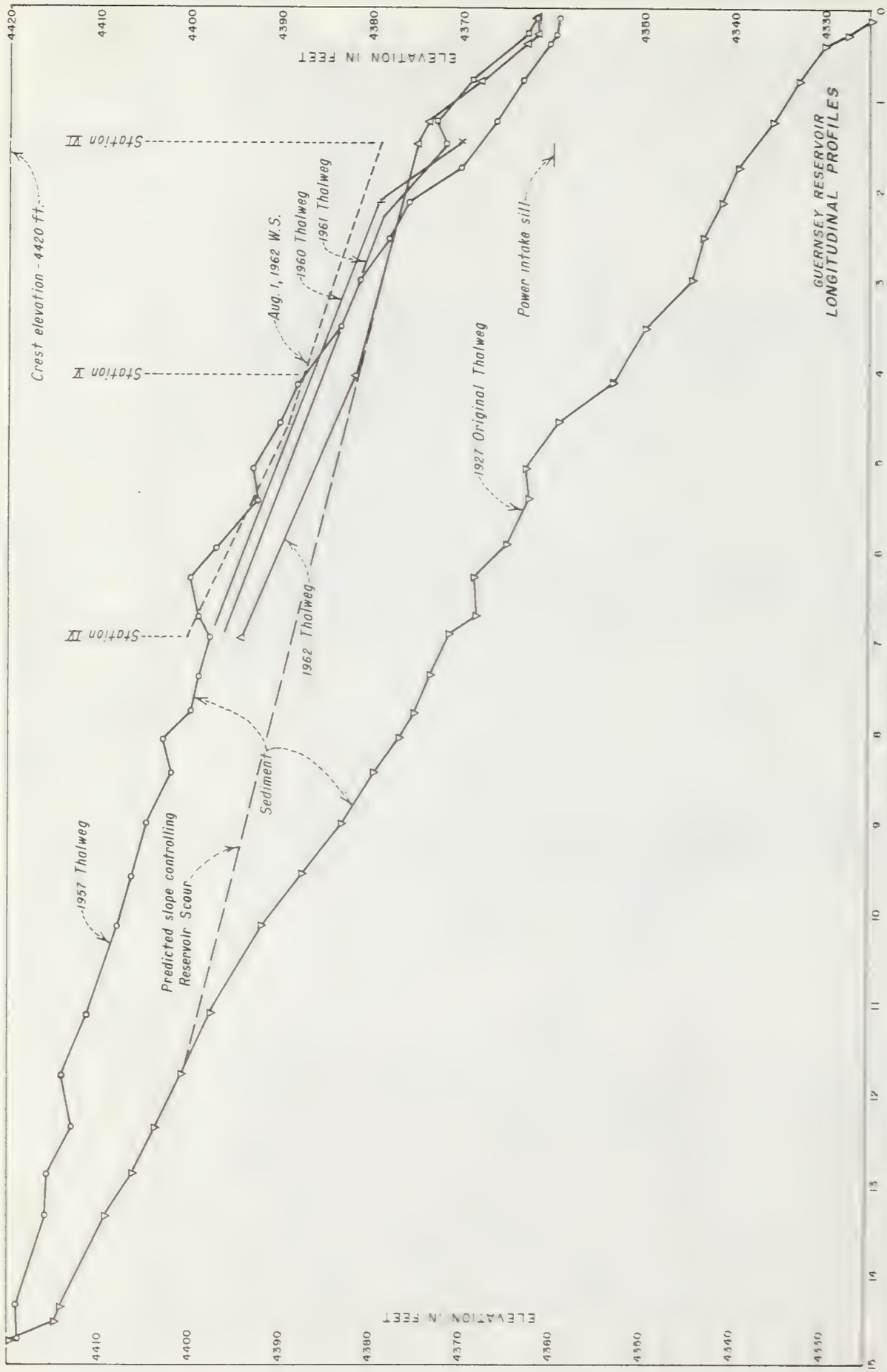

FIGURE 3. — Longitudinal profiles of sediment deposits.

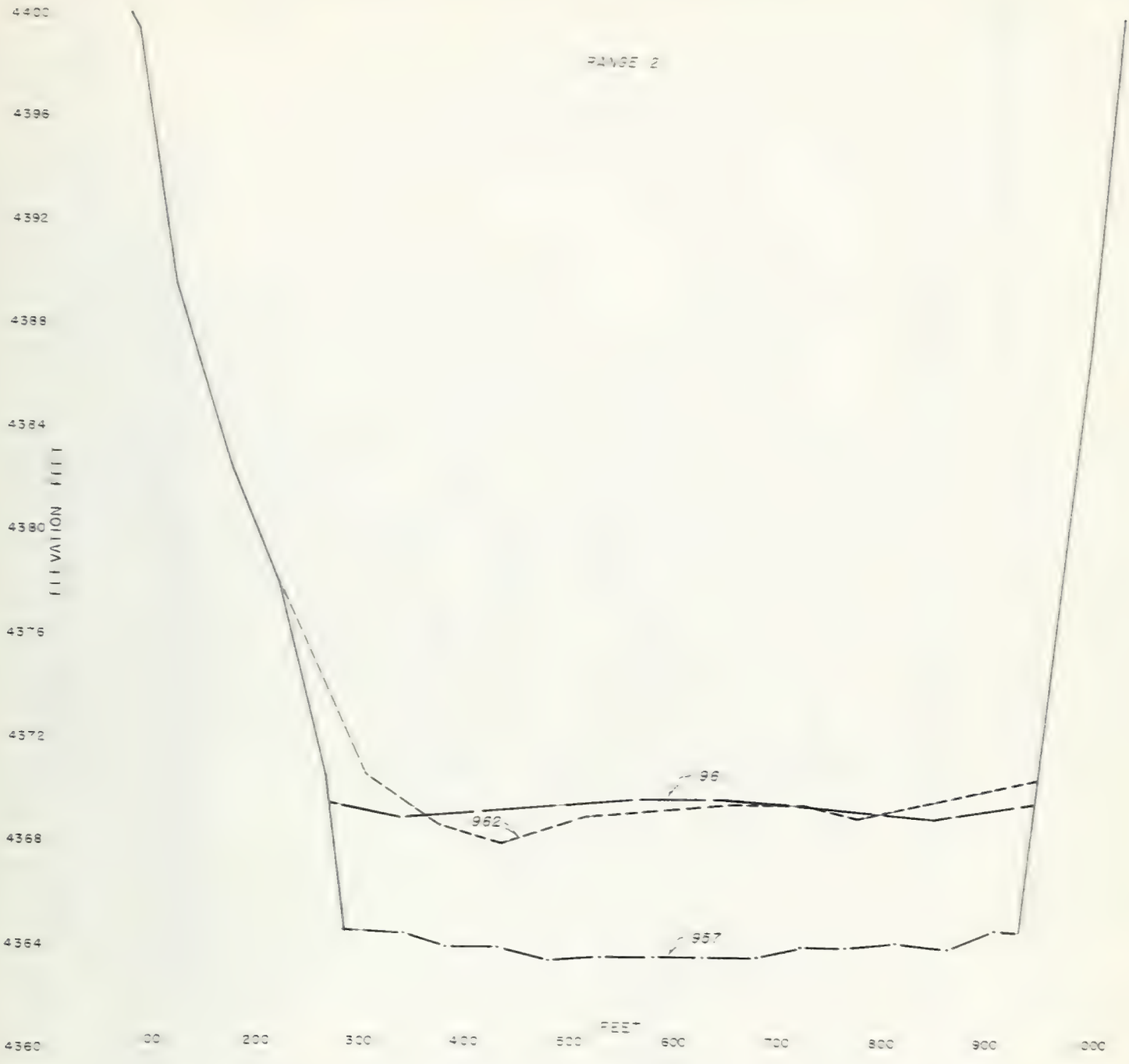

FIGURE 4.— Typical cross section of sediment deposits in lower reservoir.

capacity curves prepared from resurvey data pertaining to the sediment ranges in the reservoir study reach. The rate of drawdown had no apparent effect upon sediment concentrations in the reservoir releases, as indicated in table 3.

Effect of Storage Fluctuation

The effect of storage on the sediment concentration of reservoir releases is shown in figures 5, 6, and 7. As previously pointed out, the rates of storage fluctuation do not have significant effect on the relative efficiency; however, the period of "minimum storage" appears to be relative to increased efficiency and is certainly indicative of maximum sediment inflow and outflow. The correlation of storage to sediment concentrations of reservoir releases is shown in figure 8. A differentiation was made between sediment concentration obtained during increasing and decreasing storage. The dispersion of sediment concentration relative to increasing and decreasing storage gave no distinguishing trends in this respect. A curve was drawn representing the trend of the storage concentration relation of the plotted data. Outflow sediment concentrations relative to storage below approximately 4,000 acre-feet can have a

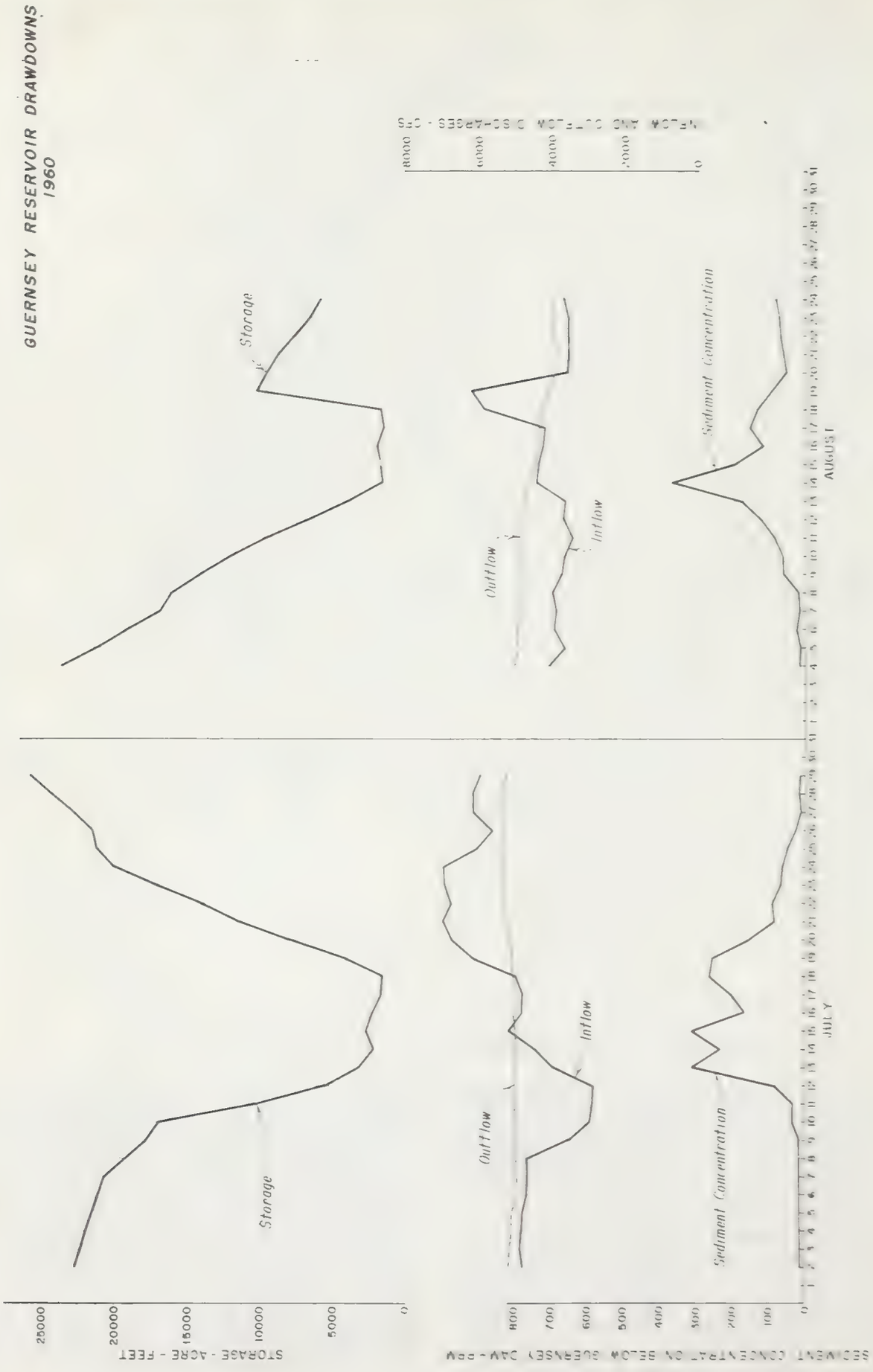

FIGURE 5. — Guernsey Reservoir drawdowns, 1960.

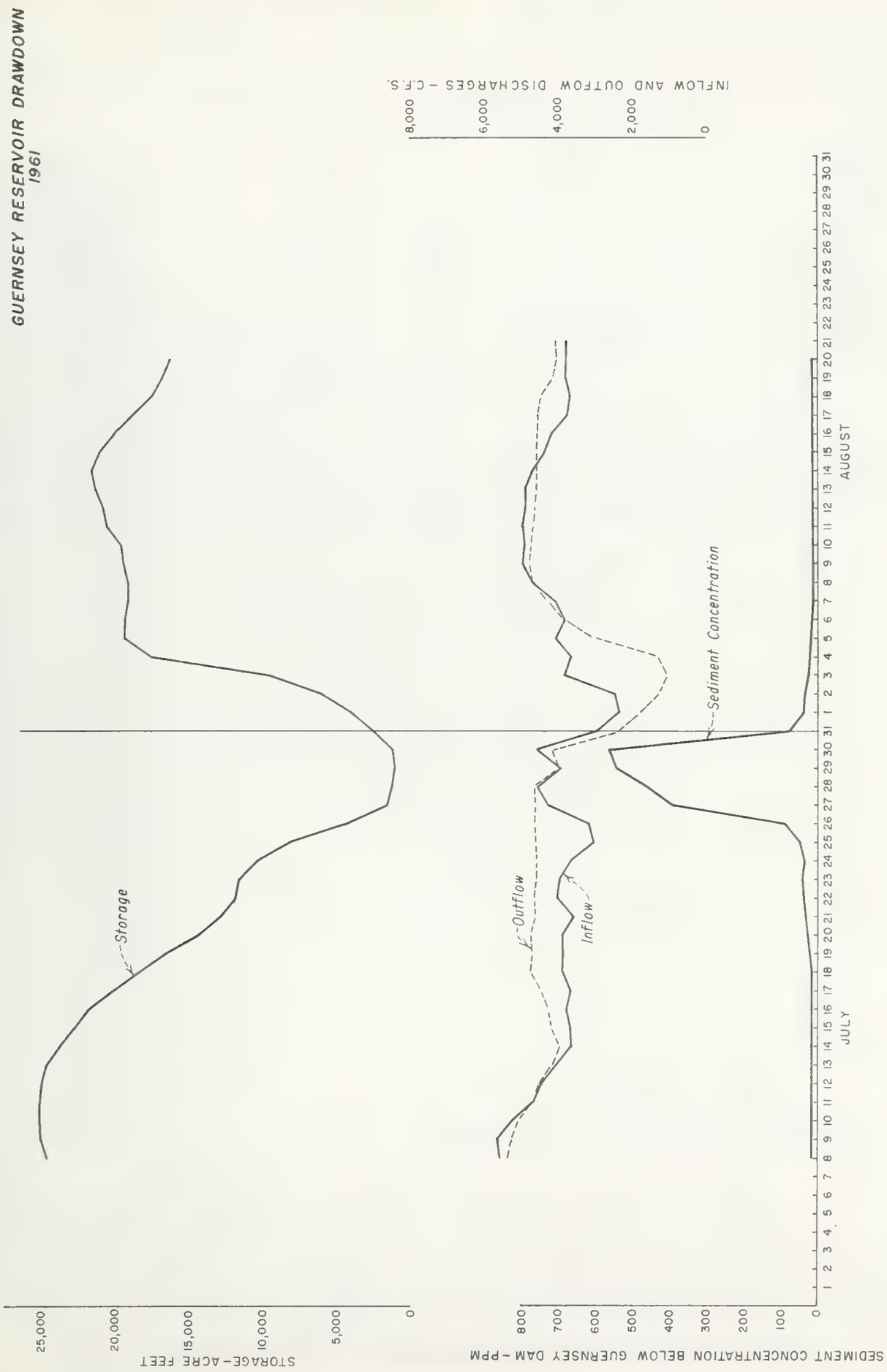

FIGURE 6. — Guernsey Reservoir drawdowns, 1961.

FIGURE 7. — Guernsey Reservoir drawdowns, 1962.

TABLE 3.—*Drawdown rates for Guernsey Reservoir, 1960–62*

Drawdown period[1]	Initial storage	Terminal storage	Change in storage	Drawdown rate	Maximum outflow concentration[2]
	Acre-feet	*Acre-feet*	*Acre-feet*	*Acre-feet/day*	*Parts per million*
July 10–12, 1960	16,900	5,500	11,400	5,700	84
Aug. 8–13, 1960	16,000	3,800	12,200	2,440	178
July 16–20, 1961	21,750	14,300	7,450	1,860	27
July 23–26, 1961	11,600	4,250	7,250	2,420	86
July 26–29, 1962	16,800	4,800	12,000	4,000	125

[1] The indicated periods were determined from the storage curves, figures 5, 6, and 7.

[2] The maximum outflow concentrations referred to in this table are the respective sediment concentrations for the terminal days of the defined drawdown periods.

relatively wide range of values for a given storage. It is also evident that maximum concentrations can be achieved with the reservoir storage below 2,000 acre-feet.

Effect of Inflow Fluctuation at Minimum Storage

A relationship to maximum outflow concentrations appears to occur between inflow fluctuation at minimum storage, as shown in figures 5, 6, and 7. At minimum storage, an increase in inflow relative to outflow results in an increase in outflow sediment concentrations. Also, when the increasing relative inflow is reversed, there is a corresponding drop in concentration. It should be emphasized that these fluctuations of the inflow are "relative to the outflow discharge" and have significant effect only at minimum reservoir storage. This phenomenon is attributed to (1) maximum exposure of sediment beds resulting in maximum inflow concentrations, and (2) a minimum quantity of water (storage) acting as a diluent on the inflow concentrations, thereby providing higher concentrations available for withdrawal.

Future Sediment Withdrawals

The source of sediment that can be scoured from Guernsey Reservoir is sediment deposited above the dam since closure of the dam in 1928. With sediment inflow now curtailed by the upstream Glendo Dam, the sediment withdrawal

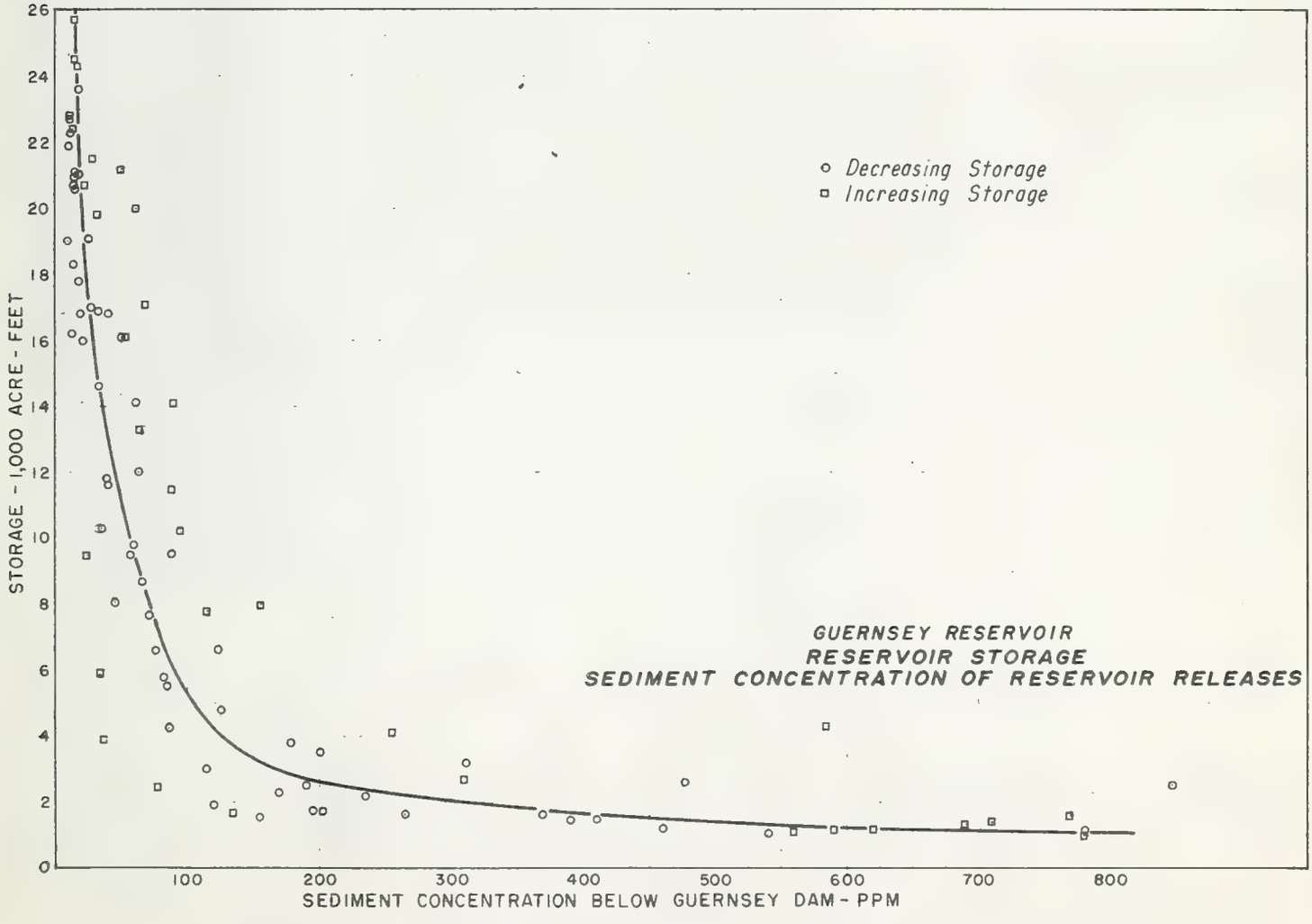

FIGURE 8. — Reservoir storage-sediment concentration of reservoir releases.

FIGURE 9. — Guernsey Reservoir range and station locations, May to July 1957.

investigations have shown that sediments can be flushed from the reservoir area. In analyzing the potential of future withdrawals from the reservoir, an appraisal was made to determine if the withdrawals of 1960, 1961, and 1962 show any decreasing rates of scour and to see if an estimate could be made of the physical limitations of future withdrawals.

Rate of Scour

Any analysis of the changing rates of sediment withdrawal from the reservoir during the 3 years of drawdown must include some appraisal of the sediment redistribution by areas within the reservoir. As shown on the reservoir topography (figs. 9, 10, and 11), there is considerable variation in location of sediment deposits within the reservoir. In the lower and upper parts of the reservoir, the deposits are confined between narrow canyons, while there is a wide flat area of deposition in the vicinity of sampling station IV and downstream to sampling station V. The suspended sediment samples collected at the sampling stations were used to compute the changes that took place during the drawdown periods. These changes in acre-feet between sampling stations are shown in table 4. Acre-feet values were determined by use of a unit weight of 67 pounds per cubic foot, which was supported by density samples of sediment deposits and a computed unit weight for deposited material of 22 percent clay, 62 percent silt, and 16 percent sand. Although it is realized the unmeasured load at each of the sampling stations would affect the results given in table 4, only those stations within the reservoir would have significant unmeasured load and the values shown should be adequate for comparative purposes.

To illustrate the results of the redistribution study, the changes shown in table 4 are plotted by reaches on figure 12. Since the volume of water varied for each drawdown period with 85,100 acre-feet for the July 10-19, 1960, period, 89,400 acre-feet for the August 8-18, 1960, period, 107,200 acre-feet for the July 20-August 3, 1961, period, and 179,000 acre-feet for the July 24-August 12, 1962, period, the changes were converted to acre-feet per acre-foot inflow for comparative purposes. In general, the relationship, as given on figure 12, shows a decreasing rate of channel scour in the upper parts of the reservoir and a decreasing rate of deposition in the lower part of the reservoir.

TABLE 4.—*Sediment redistribution resulting from reservoir drawdowns*

Reach defined by sampling stations (1)	Deposition (+) or scour (−)			
	1960		1961	1962
	July 10–19 (2)	August 8–18 (3)	July 20–August 3 (4)	July 24–August 12 (5)
	Acre-feet	*Acre-feet*	*Acre-feet*	*Acre-feet*
II to IV...	−34.4	−40.5	−29.0	−10.3
IV to V....	−166.2	−45.6	−45.5	−101.6
V to VI...	+39.3	+9.2	−21.0	−40.4
VI to Dam.	+146.5	+66.2	+78.0	+107.7
Total....	−14.8	−10.9	−17.5	−44.6

Physical Limitations

The second factor considered in estimating the future sediment withdrawals from Guernsey Reservoir is the physical limitation on scour. Three possible interrelated conditions that appear to be acting as controls on scour are (1) the reservoir storage near the dam, (2) a limiting or controlling channel slope, and (3) a limiting channel width. The longitudinal profiles on figure 3 give some indication of the effect that reservoir storage and limiting channel slope would have upon future reservoir channel scour. The only data available on ultimate channel widths through the scouring portion of the reservoir were channel widths at sampling stations observed during the drawdowns of 1960, 1961, and 1962.

The controlling slope for ultimate channel degradation in the reservoir is supported by information shown on the longitudinal profiles (fig. 3). The slope of delta deposits in the reservoir of about 2.5 feet per mile, or 0.00047 foot per foot, for the 1957 thalweg was also observed at the time of the 1937 and 1947 surveys of Guernsey Reservoir. This was also

FIGURE 10. — Aerial views of Guernsey Dam and Reservoir: (top) during reservoir drawdown of July 14, 1960; and (bottom) looking southeast toward dam during drawdown. Sediment deposits between sampling stations IV and V are in foreground.

FIGURE 11. — (Top) North Platte River flowing through sediment deposits in Guernsey Reservoir during drawdown, August 1, 1962. Sloughing unstable banks are shown in reach between sampling stations IV and V. (Bottom) Aerial view of sediment deposits in area above sampling station V during drawdown July 28, 1961. This view shows channel scour and meander within the reservoir.

about the same slope as determined for the bed profile between sampling stations V and VI in 1962. An analysis of sediment samples collected in the delta area in 1957 and of bed material samples at sampling stations IV and V both give a mean bed material size of about 0.6 millimeter. A limiting slope of about 0.00047 also coincides with low transport rates for the reach above sampling station IV and a mean bed size of 0.6 millimeter. A control on the degradation within the reservoir will be the minimum reservoir level for future drawdowns. It is estimated that any future withdrawals will be patterned after those made in 1960–62. In this case, the controlling bed elevation in the vicinity of sampling station VI will be at about elevation 4375. Based upon the above observations, a predicted profile for ultimate reservoir channel scour is shown as figure 3.

The remaining factor of defining the controlling width for future reservoir channel scour was analyzed with respect to any observed changes in widths and stable channel widths. The observed channel widths at the sampling stations in the reaches of channel above the reservoir have varied from about 190 to 250 feet, with no obvious changes with time. However, there is evidence of more bank scour in the reach between sampling stations IV and V. An ultimate channel width, based upon a limiting tractive force of 0.1 pound per square foot, a D_{75} of 0.05 millimeter, and the ultimate slope of 0.00047, would be more nearly 600 feet.

The potential estimated material available for scour in Guernsey Reservoir is 4,700 acre-feet. This represents vertical scour down to the predicted controlling slope shown on figure 3 and lateral scour in the reach in the vicinity of sampling station IV and down to sampling station V. Of the above amount, an estimated 2,000 acre-feet is in the reach between sampling station IV and V and the remaining 2,700 acre-feet are above sampling station IV.

Predicting Future Sediment Withdrawals

Although the potential volume of sediment that can be withdrawn from the reservoir is 4,700 acre-feet, the 2,700 acre-feet of material above sampling station IV is predominantly material in the sand size function. Samples collected at sampling station IV show there had been a reduction in concentration with time to about 100 p.p.m. in 1962. It is concluded from this limitation in concentration at sampling station IV and the sand material in the delta deposits above station IV, that in computing sediment withdrawal from the reservoir the 2,000 acre-feet of material between sampling stations IV and V be considered as that available for scour from the reservoir. Based upon the average size analyses of deposited material, there would be about 1,700 acre-feet of sediment in this reach of silt and clay sizes.

In predicting the future scouring capabilities in Guernsey Reservoir, a rate of scour must be correlated with the 1,700 acre-feet considered as potentially available. If drawdowns in the future can be patterned after the drawdowns of the 3 years, 1960–62, the average rates of scour as shown on figure 12 should give some indication of the future rate. A projection of the data on figure 12, combined with the estimated physical limitation of scour as shown on figure 3, would indicate an average rate of about 0.00018 acre-foot per acre-foot of water inflow for future withdrawals. With withdrawals averaging 100,000 acre-feet of inflow per period, the average sediment scoured from the reservoir would be 18 acre-feet per drawdown period.

Conclusions

There is an estimated 1,700 acre-feet of material available in Guernsey Reservoir that could be scoured during properly regulated withdrawals.

Analysis of the withdrawals made during 1960, 1961, and 1962 shows that for effective scouring of reservoir sediment deposits, the reservoir should be drawn down to 2,000 acre-feet, or below. The relationship of sediment concentration below Guernsey Dam, with storage capacity shown on figure 8, defines the

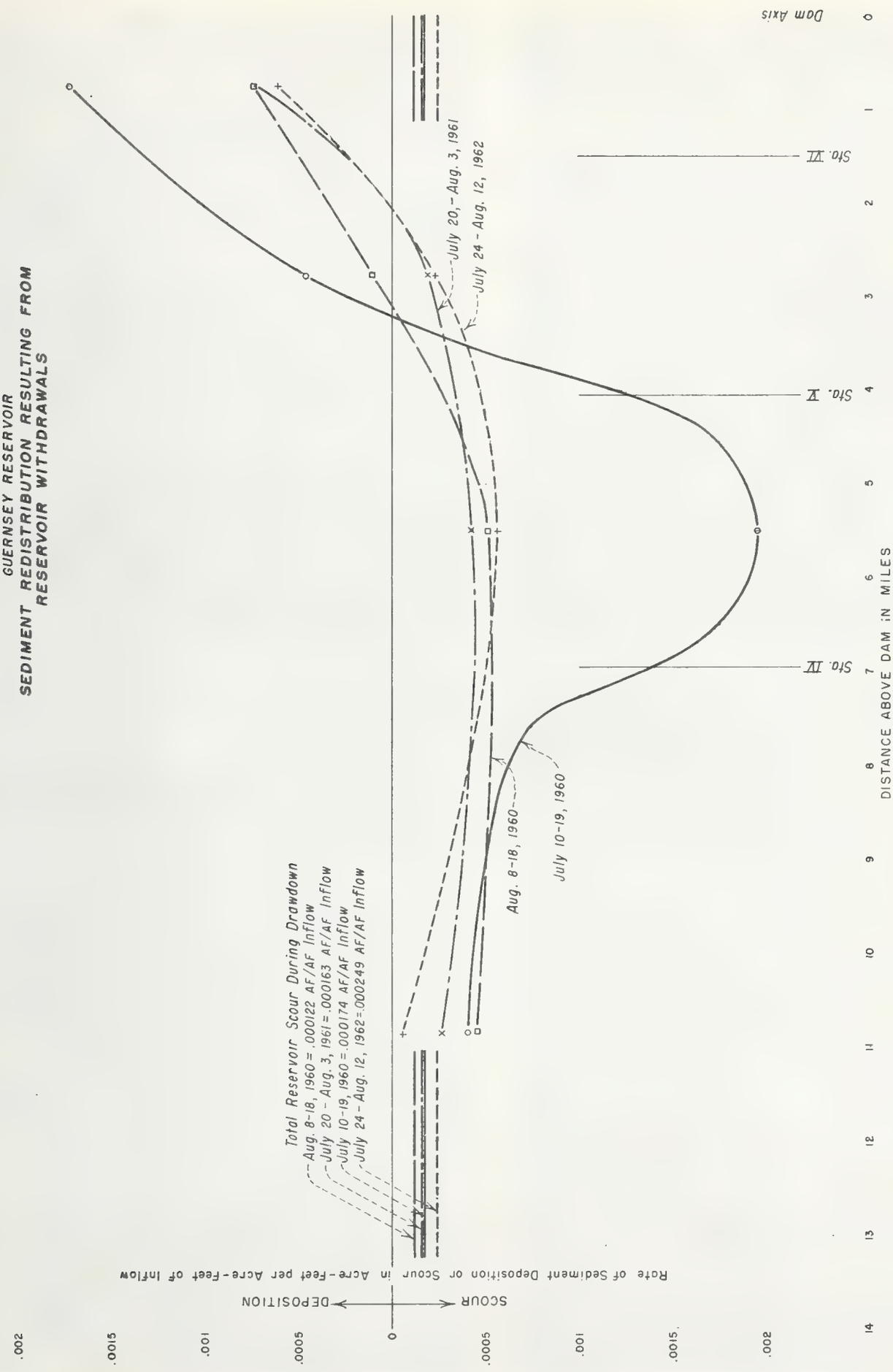

FIGURE 12. — Sediment redistribution resulting from reservoir withdrawals.

recommended storage that will result in given concentrations below the dam. This can be used to determine the best operation for obtaining the proper sediment concentration in the river relative to the requirement for the downstream distribution system.

Fluctuation of the reservoir inflow during a drawdown period appears to have an effect on sediment concentrations below Guernsey Dam. After a minimum reservoir storage has been reached, a fluctuation of about 1,000 to 2,000 c.f.s. at 1- to 2-day intervals should help maintain sediment concentration in the outflow from Guernsey Reservoir.

With drawdowns to 2,000 acre-feet capacity, or below, and by proper fluctuation of the reservoir, an estimated future sediment withdrawal rate of about 18 acre-feet appears possible for a 10- to 15-day period with an inflow of about 100,000 acre-feet. If it is assumed that sediment would be withdrawn from the reservoir twice annually for periods of 10 to 15 days each, then the estimated future annual sediment withdrawal rate would be approximately 36 acre-feet. This value, combined with an average annual withdrawal during the nondrawdown period, averaging about 6 acre-feet per year as derived for the 1957–62 period, results in an estimated total annual outflow of about 42 acre-feet. The 1,700 acre-feet of material expected to be scoured within the reservoir and the small contribution of sediment from above Guernsey Reservoir should continue to give the desired sediment outflow for about 40 years. It is anticipated that the withdrawal rate will diminish with time; however, the amount of data accumulated is not sufficient to prognosticate as to the effect of diminishing withdrawal rate.

There can be expected to be an increase in the sand size material flushed from the reservoir, which has not been considered in this analysis. An additional detrimental effect will be the increased amounts of sand, as well as finer material in water flowing through the turbines.

Acknowledgments

The authors wish to express their appreciation to W. K. Lundgreen, soil and moisture conservation engineer, Torrington, Wyo., and E. L. Pemberton, hydraulic engineer, Office of Chief Engineer, Denver, Colo., both of the Bureau of Reclamation, for their cooperation and contributions to this paper.

DISCUSSION — SYMPOSIUM 4

Sedimentation in Reservoirs

Moderator: B. L. Hobbs, CE
Panel: L. C. Gottschalk, SCS
C. R. Miller, ARS
D. C. Bondurant, CE

MR. HOBBS:

Satisfactory engineering solutions to reservoir sediment problems require reasonably accurate forecast determinations of (1) quantities of material expected to be delivered to the reservoirs; (2) amounts of material expected to be carried through the reservoirs and discharged with water releases; (3) volumes of material retained; (4) average rates of sediment accumulations in channels tributary to reservoirs; and (5) depletion of reservoir storage space allocated for various purposes in designated elevation zones. Scheduled measurements are necessary to obtain basic information as required for operational purposes and for use in testing and improving methods of forecasting sediment quantities and distribution.

With these points in mind and in view of time limitations, the panelists are requested to confine the prearranged parts of your discussions to broad considerations involved in project planning and design investigations and to provisions for periodic measurement of sediment accumulations.

MR. GOTTSCHALK:

On the subject of trap efficiency, I wish to direct attention to a definition of the term as "ability of a reservoir to retain sediment," where amounts of materials retained are expressed as percentages of the total quantities delivered to the reservoirs. The importance attached to trap efficiency in connection with project planning and design investigations has increased with the development of an entirely new retarding-type reservoir and with the increased knowledge of the interrelations of the numerous factors that affect sediment deposition and retention.

The first trap efficiency investigations were undertaken in the early 1930's by Carl B. Brown, who developed useful relations between trap efficiencies and the ratios of total reservoir capacities to drainage area sizes. However, it was found that these relations were not valid for all conditions, particularly those found in reservoirs located in the arid and semiarid areas in the United States. This led

to the investigations by Gunnar Brune, who developed a curve relating trap efficiency to the capacity-inflow ratios.

In addition, Brune attached considerable importance to (1) the "detention time" of sediment-laden water as a factor influencing deposition and (2) the related time factor involved in the decline of trap efficiency that starts at some stage during the period of storage depletion. Although investigators have recognized that numerous other factors influence trap efficiency, for the most part the early studies were based upon data obtained from single-purpose storage reservoirs where the time factors were found to be of primary importance.

New types of reservoirs have come into being as units in plans for accelerated development of water resources; these include the so-called dry reservoirs designed for operation to retard flood flows and reservoirs planned for operation to serve two or more purposes. In connection with investigations of these projects we have gradually accumulated evidence that, under certain conditions, some of the following factors affecting deposition may be equally as important as the interrelated time influences. The factors include reservoir shape; type of regulation; location of tributaries to the reservoir with respect to the outlets; sediment characteristics including mineral constituents as well as particle size; and chemical composition of the water as it affects the settling velocity of discrete particles.

In the case of the so-called "dry reservoirs," materials previously deposited tend to be "reworked" and transported farther into or out of the reservoirs than is found in the permanent-storage reservoirs. Also, stilling effects during the initial periods of inflow, which tend to cause deposition before high inflow rates exceed the release rates, are generally not present in dry reservoirs. This is usually true to some degree at all stages of a flood when releases are equal to or exceed inflow rates. To illustrate the potential importance of this stilling effect upon the trap efficiency of a dry reservoir, we should take note of a comment by Robert F. Piest stated in paper No. 15 (p. 98) that on small watersheds, ". . . storms with a return period less than one year were the cause of 34 to 92 percent of the total sediment yield."

The Soil Conservation Service is constructing a type of "upstream retarding structure" at a rate of about 700 per year. The urgent need for information regarding trap efficiency of reservoirs created by these structures has resulted in initiation of a plan for cooperative study by the Soil Conservation Service, Agricultural Research Service, and the U. S. Geological Survey, which provides for field measurements to obtain basic data. In conclusion, I suggest that further investigation be initiated to improve capabilities for estimating trap efficiency for all types of reservoirs.

MR. MILLER:

I am impressed with the coverage contained in the papers relating to sedimentation in reservoirs. This information represents an excellent documentation of the progress that has been made since the 1947 Conference, ranging from instrumentation and techniques for sediment measurement to methods developed to forecast conditions from 50 to 100 years in the future. In addition to the value of the papers as records of present knowledge and descriptions of engineering practices being used in connection with reservoir sediment problems, the papers will prove to be useful indicators of areas of need for further investigation and the proper direction of such efforts.

I consider the empirical methods for making forecast estimates of distribution of reservoir deposits, described by Heinemann,[1] and Borland and Miller,[2] to be generally satisfactory for predicting locations of the bulk of future deposits in connection with estimates of storage depletion. However, the methods presently available have many weaknesses and I consider them inadequate in several respects.

The use of reservoirs for recreational purposes is continuing at an accelerated pace, and demands for storage allocations to be used primarily for recreational activities are growing. We are becoming aware of the relations between the exposure of reservoir sediment accumulations and the number of mosquitoes. These developments have added to the importance associated with improving procedures for estimating sediment distributions corresponding to various regulation schedules. We expect probable errors in forecast estimates of distribution of the total deposits as determined by existing methods to be negligible insofar as storage depletions are concerned. However, differences in the location of relatively small fractions of the total may be quite important in connection with determinations relating to outlet designs and real estate takings. Also, drainage and ground water problems may be sensitive to small differences in sediment distribution. Papers presented by Mr. Ferral and Mr. Hadley[3] and work by other investigators indicate progress is being made in studies of channel aggradation resulting from reservoir effects.

Our procedures for estimating sediment dis-

[1] See paper No. 83, p. 845.

[2] "Distribution of Sediment in Large Reservoirs," Amer. Soc. Civil Engin. Proc. 84 (No. HY2) Paper 1587, 1958.

[3] See papers Nos. 44 (p. 376) and 80 (p. 806), respectively.

tributions could be improved; especially, we should refine procedures for predicting delta developments and progressive positions of accumulations near reservoir outlets.

MR. BONDURANT:

I believe the selection of papers has provided excellent coverage of reservoir sediment studies. These papers suggest the types and amounts of data that are required to serve various purposes; the types of surveys that should be conducted; the accuracies required in reservoir sediment measurements; and what measuring equipment and survey techniques can be used. I believe the information obtained by reconnaissance methods and spot measurements in many cases is entirely adequate for use as a qualitative indication of the extent of storage depletion, general magnitude of local problems, and as an aid to judgment concerning the need for and proper timing of more accurate instrumental surveys. The reconnaissance methods are particularly appropriate where sediment deposits are small and the primary interest concerns storage depletion. More detailed reservoir surveys are occasionally justified if the information is to serve as an aid in planning and designing other similar projects in the same region and as an aid to the optimum installation of permanent field facilities necessary for future sediment surveys in similar projects. The reliability of any survey, however, is largely dependent upon the judgment of the surveyor and, in the case of a reconnaissance type survey, it is particularly important that the assignment be given to an experienced man.

Insofar as distance measurements are concerned, there is a sufficiently wide range of good methods and techniques to allow a selection adaptable to any particular reservoir situation and type of survey one would want to make. The current meter method, first used for surveys of submerged areas by the Southwestern Division of the Corps of Engineers to determine distances of boat movement along a range line, gives good results in still water.

The use of a piano wire and reel mounted on a boat to measure distances from a shore position was first described in a TVA paper presented at the First Sedimentation Conference held in 1947. Since that time many improvements of the piano wire method provide more accurate data. These improvements include (1) the distance measuring wheel; (2) tensiometer devices to indicate the correct tension to be held in the wire; (3) floats to prevent sag; and (4) a distance-wheel attachment that automatically makes "fix" marks on a sonic depth recorder chart at intervals represented by 100 feet of boat travel, recently developed by the New Orleans District of the Corps of Engineers. This latter item is discussed in an article on pages 37 to 39 of the January–February 1962 issue of the Military Engineer. The "piano wire" method, with the wire permitted to sag to the bottom, has proved to be reasonably accurate for ranges of up to 15,000 feet. The error due to the sag is minimal on wide, shallow ranges and greater on narrow, deep sections, but a sag calculation will show that it is not generally as large as the errors inherent in other methods of distance measurement. The radist described by Isaac Shepherdson[4] is accurate and perhaps the most satisfactory position-finding equipment that could be used under reservoir conditions involving extreme distances and depths.

I believe we should continue to improve the measuring equipment, because future needs cannot be foreseen. However, in the matter of normal range-type surveys of large reservoirs, good judgment in spacing of ranges for practicable surveys is one of the more important factors. In this connection, because of irregular reservoir boundaries and economic limitations placed upon the number of ranges that can be provided, merely increasing precision of profile surveys beyond that which can be attained by existing methods would not greatly improve the overall accuracy of reservoir capacity determinations and would be of questionable justification.

There is a dearth of dependable basic data regarding the densities of reservoir deposits and rates of compaction of materials of different characteristics deposited under the various conditions and subjected to varying conditions of submergence. Sediment load records of weight quantities and volumetric estimates of erosion are helpful in reservoir sediment investigations, but the degree of confidence to be placed in forecasts depends to considerable extent upon the soundness of the engineer's judgment in predicting future densities of the reservoir deposits. Accordingly, the improvement of our capabilities for making sediment-density observations is perhaps the most urgent present need insofar as field measurements of reservoir deposits are concerned.

I believe the development of the nuclear probe is the most important advancement for measurement of in-place densities of sediment deposits that has been made in many years. Densities indicated by the probe are considered to be quite accurate, but there are numerous operational obstacles that impose limitations on the practicability of making a desirable number of density measurements. These difficulties multiply with the increase in the size of the reser-

[4] See paper No. 86, p. 869.

voir and water depths. Problems of penetrating deposits with densities exceeding 30 to 40 pounds per cubic foot are formidable, and the question regarding the extent to which the probing operation affects the density of the materials measured needs investigation.

MR. HOBBS:

We believe the foregoing statements provided adequate answers to some of the written questions, which were submitted prior to the panel session. The essence of questions directed to the panel members and authors of individual papers are presented below:

Mr. Miller, what approaches and procedures have been developed for predicting future delta formations with respect to shapes and extent of upstream deposition?

MR. MILLER:

As I stated earlier, tools for estimating future delta formations are inadequate. Some work has been done in an effort to develop step methods for predicting delta formations by applying sediment transport equations in combination with the available empirical methods used to obtain reasonable approximations of quantity distributions. This work, by employees of the Bureau of Reclamation, Corps of Engineers, and Agricultural Research Service, has not progressed to a point where publication of general predicting methods is warranted. Several papers dealing with various aspects of the delta formation problem have appeared in the literature since 1950. The paper, "Deposition at the Heads of Reservoirs," by A. S. Harrison that appeared in the Proceedings of the Fifth Hydraulics Conference, held at the State University of Iowa in June 1952, contains a comprehensive evaluation of the problem.

MR. HOBBS:

The gamma ray probe as presently developed from work originally sponsored by the Beach Erosion Board (Corps of Engineers) appears to be the best available means of determining the density of deposited reservoir sediment. The probe and accessories are expensive, however, and require special training and clearance for their use. What type of samples does the panel recommend for volume-weight determination in cases where the gamma probe is not available?

MR. BONDURANT:

A probe, of the same model as the one on display at the conference, is available at the Inter-Agency Sedimentation Project at St. Anthony Falls, Minneapolis, Minn., for loan to agencies of the U.S. Government. At present it is advisable for anyone requesting a loan of the instrument to arrange for 1 or 2 days of instruction by someone familiar with its operation. It is anticipated that when an operation manual is finally prepared, there will be no particular need for special instruction. No license is required to use the probe, and with exercise of reasonable precautions it is not dangerous. However, it is advisable to check with State health agencies regarding their position on the use of the probe.

I have no suggestion regarding a type of sample to be taken in lieu of probe measurement. Perhaps the most comprehensive information regarding instruments for obtaining in-place soil samples is contained in a report entitled "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," by M. Juul Hvorslev, and published by the Waterways Experiment Station, Vicksburg, Miss., in 1948. However, the expense involved in sampling and analyzing sediment samples obtained by the instruments described, as well as the accuracy of the results, does not compare favorably with costs and results obtained by use of the nuclear probe. I suggest that arrangements for loan of the probe is simpler than many people realize.

MR. CALDWELL:[5]

The Corps of Engineers, Baltimore District, has developed a scheme to effect probe penetration to depths from 30 to 40 feet through deposits in the Baltimore Harbor. In this technique water is pumped through perforations in pipe used to suspend the probe and force it into the deposits — jets of water emerging from the perforations above the probe flow upward and reduce friction resistance between the sides of the pipe and the sediment.

DR. BROOKS:[6]

It seems that we have made a lot of progress in finding out about sediment in reservoirs, especially in surveys and so on. I would like to ask the panel, what are we going to do about sediment in the next century or two?

MR. GOTTSCHALK:

From the standpoint of design it is the practice of Federal agencies to provide sufficient reservoir capacity for sediment storage so that deposits will not encroach upon space allocated for useful purposes, during the period used for economic analysis of the project. In general, dredging is the only effective way to "desilt" a reservoir. In most cases in this country, however, it is not economical to restore capacity by desilting a reservoir; costs involved in dredging, hauling, and providing for disposal areas are usually prohibitive. There have been many

[5] J. M. Caldwell, Beach Erosion Board.

[6] Norman Brooks, California Institute of Technology.

suggestions for desilting reservoirs, but only a few have been effective in special cases and none have general application.

MR. MILLER:

In many cases, our economic system requires that reservoirs be designed with a view to eventual obsolescence.

DR. BROOKS:

We need a little more explicit answer and we should give more thought to the prospective problems of sediment in reservoirs three or four hundred years hence.

MR. SHARP:[7]

Although we are solving some problems concerning reservoir sediment now, we are losing sight of major long range problems that will eventually confront us. We should consider solutions to the problem of holding sediment at its source, where it represents productive soil; this solution to this problem is more important than the relatively short range problems associated with sediment after it reaches the streams. For example, degradation and aggradation are important on the Lower Colorado River, but the big problem involves such questions as: keep that thing from closing when 100,000,000 acre-feet of water per year when there is nothing but a mud flat above Hoover Dam?" and "In the Columbia River, how are we going to ". . . how are we going to salvage 10,000,000 acre-feet of fresh water that goes down the river every year is diverted to the interior basins to irrigate land . . . ?"

MR. HOBBS:

The foregoing discussions represent abbreviated considerations of only a few of the problems relating to a single aspect of the sedimentation field. At the outset the problems were oriented primarily for benefit of the working engineer with an assignment. Accordingly, readers of the papers at this conference should consider this and not be misled by such limited treatment of the subject of reservoir sedimentation. For example, it should be understood that there are several reservoirs in different regions of the United States where storage depletion is expected to be negligible for long periods, ranging upward from 200 to 1,000 years, and possibly longer. In such cases it seems reasonable to expect that the structures creating some of these reservoirs probably will be obsolete before storage depletion becomes a problem.

It is obvious from the implications in the last question presented above and the statements which followed that authorities on the subject feel strongly that more thought should be given to long range plans for (1) eventual use of partially filled reservoirs; (2) provision of substitute facilities; and (3) soil conservation measures that would reduce sediment yields. The panel members and I agree generally with these ideas, but we view planning as a continuing requirement and feel that any firm plan for reservoirs that extends far beyond periods used for current economic analyses would not be in accordance with present policies of the Federal Government.

If the thoughts expressed here serve to stimulate some further interest in the field, the discussions will have served their purpose.

[7] A. L. Sharp, Agricultural Research Service.

LIST OF ATTENDANCE

U.S. DEPARTMENT OF AGRICULTURE

Agricultural Research Service

Allen, Paul B., Box 400, Chickasha, Okla.
Baird, Ralph W., P.O. Box 1147, Riesel, Texas
Bowie, Andrew J., P.O. Box 112, Holly Springs, Miss.
Chepil, William S., 2705 Browning, Manhattan, Kans.
Coleman, Neil L., P.O. Box 30, Oxford, Miss.
Dendy, Farris E., P.O. Box 30, Oxford, Miss.
Doty, Coy W., P.O. Box 165, Holly Springs, Miss.
Dvorak, Verne, 134 S. 12th Street, Lincoln, Nebr.
Glymph, Louis M., Jr., Plant Industry Station, Beltsville, Md.
Grissinger, Earl H., P.O. Box 30, Oxford, Miss.
Harrold, Lloyd, 74 Sheridan Rd., Coshocton, Ohio
Heinemann, H. G., 134 S. 12th Street, Lincoln, Nebr.
Hickok, Robert B., P.O. Box 2326, Riverside, Calif.
Kelly, L. L., Plant Industry Station, Beltsville, Md.
Keppel, R. V., P.O. Box 3926, Tucson, Ariz.
Long, Fred W., 108 Soil Science Bldg., University of Minnesota, St. Paul 1, Minn.
McDowell, L. L., P.O. Box 30, Oxford, Miss.
McHenry, J. Roger, P.O. Box 30, Oxford, Miss.
Martin, Walter W., Plant Industry Station, Beltsville, Md.
Miller, Carl R., P.O. Box 30, Oxford, Miss.
Minshall, Neal, 1508 University Avenue, Columbia, Mo.
Parsons, Donald A., P.O. Box 30, Oxford, Miss.
Piest, Robert F., P.O. Box 30, Oxford, Miss.
Ree, William O., P.O. Box 789, Stillwater, Okla.
Ross, Alex R., Dept. of Geology, Oklahoma State University, Stillwater, Okla.
Sharp, A. L., 134 S. 12th Street, Lincoln, Nebr.
Stein, Richard A., P.O. Box 30, Oxford, Miss.
Stephens, John C., P.O. Box 1309, Athens, Ga.
Whelan, Donald E., Old Orchard Road, Silver Spring, Md.
Wischmeier, Walter H., 2368 N. 25th Street, Lafayette, Ind.
Woodburn, Russell, P.O. Box 1309, Athens, Ga.
Yates, Paul, P.O. Box 33, Watkinsville, Ga.

Forest Service

Anderson, Henry W., P.O. Box 245, Berkeley, Calif.
Baker, Robert M., 1712 Gibraltar Dr., Jackson, Miss.
Copeland, Otis L., Jr., 1470 Marilyn Drive, Ogden, Utah
Courtenay, John H., 3027 Delmonico Dr., Decatur, Ga.
Dortignac, Edward J., Rm. 4209 South Bldg., Washington 25, D.C.
Fredriksen, Richard L., P.O. Box 160, Blue River, Oreg.
Gerred, Don L., 50 7th Street, NE., Atlanta 23, Ga.
Hibbert, Alden Ron, Franklin, N.C.

Hobba, Robert L., Route 2, Frederick, Md.
Krammes, Jay S., 110 N. Wabash Ave., Glendora, Calif.
Lull, Howard W., 102 Motors Ave., Upper Darby, Pa.
MacNaughton, V. B., Oxford, Miss.
Murphy, Warren T., 6207 Crathie Lane, Washington 16, D.C.
Rice, Raymond M., 110 N. Wabash, Glendora, Calif.
Stephenson, George K., 53 Allard Blvd., New Orleans, La.
Striffler, W. David, 119½ S. Mitchell St., Cadillac, Mich.
Thompson, J. Robert, 1227 City Park Ave., Fort Collins, Colo.
Ursic, Stanley J., Oxford Research Center, Oxford, Miss.
Varney, Gilbert L., 6816 Market Street, Upper Darby, Pa.

Soil Conservation Service

Abrams, W. J., 1005 Rolling Green Drive, Waukesha, Wis.
Brown, Carl B., Agricultural Research Center, Washington 25, D.C.
Cole, William A., Box 610, Jackson, Miss.
Culp, Melvin M., 1125 Cresthaven Dr., Silver Spring, Md.
Doubt, Paul D., 511 Elderwood Road, Silver Spring, Md.
Flaxman, Elliott M., 209 S. W. 5th Street, Portland, Oreg.
Geiger, A. F., 1121 E. Vienna Ave., Milwaukee, Wis.
Goodman, Max R., New Albany, Miss.
Gottschalk, L. C., 719 Lakeview Drive, Falls Church, Va.
Holeman, John N., 2716 Crest Ave., Cheverly, Md.
Hunt, James L., 410 Candlewood Rd., Broomall, Pa.
Jacobson, Paul, 4th Floor, Iowa Bldg., Des Moines, Iowa
Lester, H. H., 1915 Terry Road, Jackson, Miss.
Maner, Sam B., 3608 McCart, Fort Worth, Tex.
Renfro, Graham W., 1531 Enderly Place, Fort Worth, Tex.
Roehl, John W., P.O. Box 613, Spartanburg, S.C.
Rowe, Lawrence A., P.O. Box 759, State College, Miss.
Thorp, E. M., 134 S. 12th Street, Lincoln, Nebr.
Ward, L. J., Box 610, Jackson, Miss.
Watts, W. L., New Albany, Miss.

DEPARTMENT OF THE ARMY
Corps of Engineers

Bayley, Fred, Vicksburg District, 920 Polk Street, Vicksburg, Miss.
Billard, Donald P., 30 Pryor Street, SE., Atlanta, Ga.
Black, William L., 30 Pryor Street, SE., Atlanta, Ga.
Blazek, Harry J., 4112 Moss Rose Drive, Nashville 6, Tenn.
Boland, Robert A., Jr., 211 Hillside Drive, Vicksburg, Miss.
Bondurant, D. C., 4101 Pratt Street, Omaha, Nebr.
Bottorf, William L. D., 630 Sansome Street, San Francisco, Calif.
Brodbeck, Ernest E., 90 Church Street, New York 7, N.Y.
Brown, Fred R., P.O. Box 631, Vicksburg, Miss.
Browne, William H., Jr., 413 E. Main, Cincinnati, Ohio
Buhler, Thornton J., Foot of Prytania Street, New Orleans, La.
Burke, Mrs. Pearl Pierce, 2829 Palmer Avenue, New Orleans 18, La.
Bywater, Harold A., Jr., 663 Sizeler Avenue, New Orleans 21, La.
Caccese, Lewis, Custom House, Philadelphia, Pa.
Caldwell, Joseph M., Beach Erosion Board, 5201 Little Falls Rd., Washington 16, D.C.
Campbell, F. B., P.O. Box 631, Vicksburg, Miss.
Cannon, James C., City-County Airport, Walla Walla, Wash.
Carey, Walter C., Foot of Prytania Street, New Orleans, La.
Cassidy, William C., Rm. 1216, 630 Sansome Street, San Francisco, Calif.
Castle, Glenn, 1209 8th Street, Sacramento, Calif.
Chew, Arthur, 517 Gold, SW., Albuquerque, N. Mex.
Christensen, Russell P., St. Anthony Falls Hydr. Lab., Minneapolis, Minn.
Cochran, Albert L., 5207 Baltimore Ave., Washington 16, D.C.
Cox, Robert G., WES, Vicksburg, Miss.
Cozzani, Anthony, 3508 Washington Street, Vicksburg, Miss.
Cummins, Reid S., Jr., WES, Box 631, Vicksburg, Miss.
Davis, John P., Rm. 1323, Bldg. T-7, Gravelly Point, Va.
Dowdy, Arvil S., SW Div., Dallas, Tex.
Drago, Emanuel A., 4720 Oxford Drive, Mobile, Ala.
Draper, Elmond L., 100 W. Vickery, Fort Worth, Tex.
Druml, Frank V., P.O. Box 59, Louisville 1, Ky.
Duncan, Walter W., 2508 Turnagain Blvd., E., Anchorage, Alaska
Feagin, L. B., Mississippi River Commission, Vicksburg, Miss.
Fisackerly, George M., WES, Box 631, Vicksburg, Miss.
Fortson, E. P., Jr., WES, Vicksburg, Miss.
Franco, John J., 161 Cane Ridge Dr., Vicksburg, Va.
Gamel, Weldon M., 303 Black Oak, La Marque, Tex.
Gates, Verne W., 180 New Montgomery Street, San Francisco, Calif.
Geelan, Charles W., 2914 Patricia Drive, La Marque, Tex.
Gildea, A. P., 751 S. Figueroa Street, Los Angeles, Calif.
Glover, J. E., WES, Vicksburg, Miss.
Graham, Jimmie, P.O. Box 60, Vicksburg, Miss.
Gravely, Walter M., 2301 Airport Blvd., Mobile, Ala.
Haas, Raymond H., Lower Miss. Div., Vicksburg, Miss.
Hamilton, John T., 4620 R. 112, Galveston, Tex.
Harris, John W., 200 E. St. Julian Street, Savannah, Ga.
Hartzell, E. P., U.S. Custom House, Philadelphia, Pa.
Henley, John E., P.O. Box 60, Vicksburg, Miss.
Herrmann, Frank E., WES, Vicksburg, Miss.
Hill, Albert L., 4117 Flora, Kansas City, Mo.
Hobbs, Brice L., 1075 N. Montana St., Arlington, Va.
Holliday, William Clyde, 5th Ave. and 8th St., Huntington, W. Va.
Hough, Leonard, 6941 Alpha Road, Dallas, Tex.
Hyde, George E., 628 Pittock Block, Portland, Oreg.
Johnson, Oliver, 210 Custom House, Portland, Oreg.
Kelley, George C., 1248 S. Oswego, Tulsa, Okla.
Leach, Laurence G., South Atlantic Div., Atlanta, Ga.
Linder, Walter M., 4669 Pine Street, Omaha 6, Nebr.
Lindner, C. P., 2276 Virginia Place, NE., Atlanta 5, Ga.
Livesey, Robert H., 1018 N. 75th Street, Omaha, Nebr.
Lockett, John B., 6323 N.W. Manor Drive, Portland 10, Oreg.
Long, Edwin G., Jr., 308 Custom House, Wilmington, N.C.
McGee, H. C., Jackson, Miss.
McKellar, C. D., Jr., P.O. Box 631, Vicksburg, Miss.
Mack, Frank J., Clock Tower Bl., Rock Island, Ill.
Madden, Edward B., 10109 McCree Road, Dallas, Tex.
Maher, Thomas F., 420 Locust, St. Louis, Mo.

Martin, John T., 2804 Utah, NE., Albuquerque, N. Mex.
Mauriello, Louis J., 90 Church Street, New York, N.Y.
Mitchell, John T., Jr., 118 Brandenberg Drive, Loveland, Ohio
Mittendorf, George H., 2662 Sharondale Circle, Atlanta, Ga.
Moreman, LeRoy B., 2301 Airport Blvd., Mobile, Ala.
Morrow, Forrest B., 6112 Black Oak Lane, Fort Worth 14, Tex.
Murphy, Thomas E., P.O. Box 631, Vicksburg, Miss.
Neat, Leyden K., 2431 Ashwood Drive, Louisville, Ky.
Northrop, William L., 911 Walnut Street, Kansas City 14, Mo.
Ord, Melvin J., Air Base, Walla Walla, Wash.
Otto, William C., 150 Michigan, Detroit 26, Mich.
Petersen, Margaret S., 5505 Scenic Drive, Little Rock, Ark.
Phillips, Joseph F., Custom House, Philadelphia, Pa.
Pickett, Ellis B., WES, Vicksburg, Miss.
Powell, Sidney S., 2301 Airport Blvd., Mobile, Ala.
Price, George A., Jr., New Orleans, La.
Randall, William J., 1519 Alaskan Way, S., Seattle, Wash.
Reese, Howard O., 616 S. Boston Ave., Tulsa, Okla.
Resta, Rodney H., 6111 McIntyre Rd., Rockville, Md.
Rhodes, Henry J., WES, Vicksburg, Miss.
Rogal, Reuben E., 564 Forbes Ave., Pittsburgh 19, Pa.
Saylor, James B., 2439 23rd Ave., Rock Island, Ill.
Seale, William B., 4038 Jefferson Highway, New Orleans 21, La.
Shamblin, O. H., Vicksburg Dist., Vicksburg, Miss.
Shamburger, John H., WES, Vicksburg, Miss.
Shepherdson, Isaac, 103 2d Street, Riverdale, N. Dak.
Simmons, H. B., WES, Vicksburg, Miss.
Smith, Austin B., Mississippi River Commission, Vicksburg, Miss.
Stanley, Lt. Daniel G., WES, Vicksburg, Miss.
Sugden, Harry, 429 E. 51, Savannah, Ga.
Tallant, Irby C., WES, Vicksburg, Miss.
Tatum, Fred E., 751 S. Figueroa, Los Angeles, Calif.
Tiefenbrun, A. J., 420 Locust Street, St. Louis 2, Mo.
Tiffany, Joseph B., WES, Vicksburg, Miss.
Toffaleti, Fred B., Mississippi River Commission, Vicksburg, Miss.
Toy, Harold W., P.O. Building, St. Paul, Minn.
Trainor, Charles F., 539 E. 57th Street, Savannah, Ga.
Watts, George M., 5201 Little Falls Rd., NW., Washington, D.C.
Weller, Harvill E., 1656 Glenwood Place, Vicksburg, Miss.
Williams, E. J., Jr., 46 Chapel Hills Drive, Vicksburg, Miss.
Witzigman, Fred S., St. Anthony Falls Hydr. Lab., Minneapolis, Minn.
Yaeger, O. B., 700 West Capitol, Little Rock, Ark.
Zander, Dean M., 5th and Gold, Federal Bldg., Albuquerque, N. Mex.

ATOMIC ENERGY COMMISSION

Thatcher, L. L., U.S. Atomic Energy Commission, Washington 25, D.C.

U.S. DEPARTMENT OF COMMERCE

Coast and Geodetic Survey

Burns, Robert E., 4427 Bradley Lane, Chevy Chase 15, Md.

Bureau of Public Roads

Herr, Lester A., 1717 H Street, NW., Washington 25, D.C.
Izzard, Carl F., Washington 25, D.C.
Searcy, James K., 1717 H Street, NW., Washington 25, D.C.
Schneible, D. E., 828 Peachtree-Seventh Bldg., Atlanta 23, Ga.

U.S. DEPARTMENT OF THE INTERIOR

Geological Survey

Albert, C. D., 527 Berkshire, Salina, Kans.
Ames, Frank C., 8000 W. 41st Ave., Wheat Ridge, Colo.
Benedict, Paul C., 345 Middlefield Rd., Menlo Park, Calif.
Beverage, Joseph P., P.O. Box 4217, Albuquerque, N. Mex.
Cabell, Roy E., Box 2657, Salt Lake City, Utah
Carter, Rolland W., 432 Shadeland Drive, Falls Church, Va.
Collier, Charles R., 2822 E. Main Street, Columbus 9, Ohio
Culbertson, Don M., 2616 Woodsdale Blvd., Lincoln, Nebr.
Curtis, William F., 1240 Gretta Street, Albuquerque, N. Mex.
Dowdy, Arvil S., SW Div., Dallas, Texas
Emmett, William W., Washington 25, D.C.
Flint, Russell F., 2822 E. Main Street, Columbus 9, Ohio
George, John R., 100 N. Cannon Street, Harrisburg, Pa.
Geurin, James W., Rm. 1252 Fed. Bldg., 1520 Market St., St. Louis, Mo.
Golden, Harold G., P.O. Box 2052, Jackson, Miss.
Guy, Harold P., Colo. State Univ., Fort Collins, Colo.
Hadley, Richard F., Bldg. 25, Fed. Center, Denver, Colo.
Hanly, Thomas F., 1214 Bighorn Avenue, Worland, Wyo.
Haushild, William L., 830 NE. Holladay, Portland 8, Oreg.
Hubbell, David W., Colorado State University, Fort Collins, Colo.
Jones, Benjamin L., 100 N. Cameron Street, Harrisburg, Pa.
Jopling, Alan V., Boston, Mass.
Kapustka, Stanley F., 221 E. Parkland Drive, Baton Rouge, La.
Keller, Frank J., 2010 Rockland Avenue, Rockville, Md.
Love, S. K., Washington 25, D.C.
Lusby, Gregg C., Bldg. 25, Federal Center, Denver, Colo.
Maddock, Thomas, Jr., Box 4070, Tucson, Ariz.
Mundorff, James, Rm. 513, Rudge & Guenzel Bldg., Lincoln, Nebr.
Murphy, John J., 1209 Fernandez Juncos, San Juan, P.R.
Musser, John J., 2822 E. Main Street, Columbus 9, Ohio
Nordin, Carl F., Jr., P.O. Box 4217, Albuquerque, N. Mex.
Pickering, R. J., P.O. Box 1065, Oak Ridge, Tenn.
Porterfield, George, 650 Capital Avenue, Rm. 8542, Sacramento 21, Calif.
Reeder, Howard E., Federal Bldg., Raleigh, N.C.
Riggs, H. C., 2427 Executive Avenue, Falls Church, Va.
Schumm, Stanley A., 2783 S. Stuart, Denver, Colo.
Scott, Cloyd H., 513 Rudge & Guenzel Bldg., Lincoln, Nebr.
Shattles, Donald E., 244 Federal Bldg., Ocala, Fla.
Shell, James D., P.O. Building, Jackson, Miss.
Simons, D. B., 270 Circle Dr., Fort Collins, Colo.
Stephens, Howard D., Rm. 513, Rudge & Guenzel Bldg., Lincoln, Nebr.
Stow, J. M., 5516 Arvilla Ave., NE., Albuquerque, N. Mex.
Vice, Raymond B., Washington 25, D.C.
Wark, John W, 3 N. Perry St., Rockville, Md.
White, Walter F., 333-W Arlington Towers, Arlington 9, Va.

Williams, Robert C., 830 N. E. Holladay, Portland 8, Oreg.
Wilson, Kenneth V., P.O. Box 2052, Jackson, Miss.
Wolman, M. Gordon, 2104 W. Rogers Avenue, Baltimore 9, Md.

Bureau of Mines

Berk, A. A., College Park, Md.

Bureau of Reclamation

Blanchard, Bruce, 1001 N. 1st Street, Phoenix 4, Ariz.
Buettner, LeRoy J., Box 392, Oklahoma City, Okla.
Carlson, Enos J., 1189 Racine Street, Denver 8, Colo.
Enger, Phillip "F", 5975 Garrison, Arvada, Colo.
Gessel, Clyde D., 2040 Lambourne Avenue, Salt Lake City 9, Utah
Illk, Frank K., Boulder City, Nev.
Jarecki, Eugene A., Fed. Center, Bldg. 46, Denver 25, Colo.
Lord, Robert C., P.O. Box 1609, Amarillo, Tex.
Munson, Wendell C., 1101 8th Street, Apt. A–1, Alexandria, Va.
Oliver, Paul A., Boulder City, Nev.
Pemberton, Ernest L., Federal Center, Denver 25, Colo.
Sheppard, John R., Rm. 66 N, Bldg. 53, Federal Center, Denver 25, Colo.
Terrell, Pete W., Federal Center, Denver, Colo.

U.S. DEPARTMENT OF THE NAVY

Hamilton, C. L., 4528 43rd Street, NW., Washington 16, D.C.
Keller, George H., USN Oceanographic Office, Washington, D.C.
Terbush, Leo S., S.E. Div. Bureau Docks, 1205 Montague, N. Charleston, S.C.

U.S. DEPARTMENT OF HEALTH, EDUCATION, AND WELFARE

Public Health Service

Bullard, William E., Jr., 570 Pittock Block, Portland 5, Oreg.
Rainwater, Frank H., Washington, D.C.
Rouse, Jim V., 1819 W. Pershing Road, Chicago, Ill.
West, A. W., 4676 Columbia Parkway, Cincinnati 26, Ohio

TENNESSEE VALLEY AUTHORITY

Betson, Roger, 711-A Union Building, Knoxville, Tenn.
Marius, John B., 711-A Union Bldg., Knoxville, Tenn.
Shereda, David T., TVA, Knoxville, Tenn.
Snyder, Willard M., 4123 Fulton Road, Knoxville 18, Tenn.
Sodemann, Paul C., TVA, Knoxville, Tenn.
Tysinger, Jones E., Knoxville, Tenn.

STATES AND MUNICIPALITIES

Beard, Vernon Miller, Pennsylvania Department of Forests and Waters, New Cumberland, Pa.
Cook, Herbert M., Texas Water Commission, Austin, Tex.
Corinth, Roger L., Illinois State Water Survey, Urbana, Ill.
Durbrow, William, Jr., California Department of Water Resources, Sacramento, Calif.
Ferrell, William R., Los Angeles County Flood Control District, Los Angeles, Calif.
Henderson, Ted. E., International Boundary and Water Commission, El Paso, Tex.
Meier, Wilbur L., Jr., Texas Water Commission, Austin, Tex.
Pepper, Jack W., Mississippi Board of Water Commissioners, Jackson, Miss.
Rich, Eldred, Iowa Natural Resources Council, West Des Moines, Iowa.
Stall, John B., Illinois Water Survey, Urbana, Ill.
Taylor, Ernest J., Louisiana Department of Public Works, Baton Rouge, La.

UNIVERSITIES

Anderson, James H., Mississippi State University, State College, Miss.
Beer, Craig E., Iowa State University, Ames, Iowa
Brooks, Norman H., California Institute of Technology, Pasadena, Calif.
Bruun, Per, University of Florida, Gainesville, Fla.
Dickerson, W. H., West Virginia University, Morgantown, W. Va.
Drinker, Philip A., Massachusetts Institute of Technology, Cambridge, Mass.
Einstein, H. A., University of California, Berkeley, Calif.
Falcon, MarFcd., Massachusetts Institute of Technology, Cambridge, Mass.
Franzini, Joseph B., Stanford University, Stanford, Calif.
Johnson, J. W., University of California, Berkeley, Calif.
Karaki, Susumu, Colorado State University, Fort Collins, Colo.
Kennedy, John F., Massachusetts Institute of Technology, Cambridge, Mass.
Kimbrough, E. A., Jr., Mississippi State University, State College, Miss.
Krone, Ray B., University of California, Berkeley, Calif.
Laursen, Emmett M., University of Arizona, Tucson, Ariz.
McConnell, C. H., Pennsylvania State University, University Park, Pa.
McWhorter, John C., Mississippi State University, State College, Miss.
Masch, Frank D., University of Texas, Austin, Tex.
Moore, Walter L., University of Texas, Austin, Tex.
Mostafa, Elasyouti I., Cairo University, Giza, Egypt.
Shelton, C. H., University of Tennessee, Knoxville, Tenn.
Shulits, Sam, Pennsylvania State University, University Park, Pa.
Smerdon, Ernest T., Texas A & M College, Austin, Tex.
Smith, David Dwyer, Louisiana State University, Baton Rouge, La.
Tanner, W. F., Florida State University, Tallahassee, Fla.
Vanoni, Vito A., California Institute of Technology, Pasadena, Calif.
Yevdjevich, V. M., Colorado State University, Fort Collins, Colo.

OTHERS

Aly, Hassan Wahbi M., Ministry of Public Works, Egypt
Bitoun. Marcel, Harza Engineering Co., 400 W. Madison, Chicago, Ill.
Ferrell, Neil J., Michael Baker, Inc., Northview Dr., Jackson, Miss.
Gonyaw, Nelson L., Pickering Engineering Assoc., Inc., 3120 Summer Ave., Memphis, Tenn.
Louie, David S. H., Harza Engineering Co., Riverside Plaza, Chicago 6, Ill.
Mao, Shieh-Wen, Tipton & Kalmbach, Inc., 831 14th Avenue, Denver, Colo.
Montaser, Abdel Latif, Ministry of Public Works, Egypt
Perkins, Richard W., Hanford Atomic Products Operations, Richland, Wash.
Rice, Leonard, 831 14th Street, Denver 2, Colo.
Shen, H. W., Harza Engineering Co., 400 W. Madison, Chicago, Ill.

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